UNITED STATES DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

DESIGN OF SMALL DAMS

UNITED STATES DEPARTMENT OF THE INTERIOR, Fred A. Seaton, *Secretary*

BUREAU OF RECLAMATION, Floyd E. Dominy, *Commissioner*

DESIGN OF SMALL DAMS

First Edition, 1960

UNITED STATES GOVERNMENT PRINTING OFFICE

WASHINGTON : 1960

For sale by the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., and the Bureau of Reclamation, Denver Federal Center, Denver, Colorado, Attention: 841. Price $6.50

Shadow Mountain Dam and spillway on the Colorado River in Colorado. 7–400–192.

Preface

THIS BOOK presents instructions, standards, and procedures for use in the design of small dams. It is intended to serve primarily as a guide to safe practices for those concerned with the design of small dams in public works programs in the United States. The book will serve this purpose in three ways: (1) It will provide engineers with information and data necessary for the proper design of small dams; (2) it will provide specialized and highly technical knowledge concerning the design of small dams in a form that can be used readily by engineers who do not specialize in this field; and (3) it will simplify design procedures for small earthfill dams.

An earlier publication, "Low Dams," which was prepared in 1938 by the National Resources Committee, presented much useful information on the design of small dams. In the 20 years that have elapsed since the printing of that book, however, there have been many technical advances in the design of dams, and the need for a new work incorporating the latest design techniques has become increasingly evident. It is believed that this book, "Design of Small Dams," will fill that need. The new book retains much of the format of "Low Dams" and some of the material from the earlier publication has been incorporated in the new one, but most of the text is wholly new.

Although this text is related almost exclusively to the design of small dams and appurtenant structures, it is important that the designer be familiar with the purposes of the project, the considerations influencing its justification, and the manner of arriving at the size and type of structure to be built. For these reasons an outline discussion of a desirable project investigation has been included in chapter I.

Only the more common types of small dams now being constructed are discussed. These include concrete gravity, earthfill (rolled-type), and rockfill dams. Emphasis is placed on the design of rolled earthfill dams because they are the most common type. For the purpose of this book, small dams include those structures with heights above streambed not exceeding 50 feet except for concrete dams on pervious foundations. For the latter structures, the maximum height is further limited to dams whose maximum net heads (headwater to tailwater) do not exceed 20 feet. The text is not intended to cover dams of such large volumes that significant economies can be obtained by utilizing the more precise methods of design usually reserved for large dams. In recognition of the limited engineering costs justified for small dams, emphasis is placed on efficiency and relatively inexpensive procedures to determine the necessary design data. Simplified design methods are given to avoid the complex procedures and special investigations required for large dams or for unusual conditions. Adequate but not unduly conservative factors of safety are used in the simplified design methods.

Small dams are properly considered to be associated with small streams and drainage areas of limited extent. For these situations or for those in which spillway capacity is obtainable at relatively low cost, a sufficient approximation of the inflow design flood discharge may be determined by procedures given in this text. For important projects, particularly where the spillway cost is a major item of project cost and thus may have an important bearing on project feasibility, more exact and complex studies which are beyond the scope of this text may be justified.

This text is addressed to the designer of the

structure and does not include in its scope the field of construction practices or methods. However, as the integrity of the design requires adherence to limiting specifications for materials and to the practice of good workmanship in construction, appendixes are included on "Construction of Embankments," "Concrete in Construction," and "Sample Specifications." More detailed specifications will be required to insure proper construction of any specific dam.

This text is not intended in any way to encourage assumption of undue responsibility on the part of unqualified personnel, but rather to point out the importance of specialized training and to stimulate wider use of technically trained and experienced consultants.

This text should be of service to all concerned with the planning of small water storage projects, but in no way does it relieve any agency or person using it of the responsibility for safe and adequate design. The stated limitations of the design procedures should be heeded.

This book was prepared by the engineers of this Bureau, United States Department of the Interior, in my office at Denver, Colo., under the direction of Grant Bloodgood, Assistant Commissioner and Chief Engineer, and L. G. Puls, Chief Designing Engineer. More than 30 engineers and many technicians participated in the preparation of the book or in its critical review, and the efforts of all of these are gratefully acknowledged. Special recognition is given O. L. Rice, Chief of the Dams Branch, for his guidance and counsel, especially in determining the scope and treatment of the text.

The text was coordinated and edited by H. G. Arthur, Supervisor, Design Unit, Earth Dams Section, and final review and preparation of the manuscript for the printer was by E. H. Larson, Head, Manuals and Technical Records Section.

The Bureau of Reclamation expresses grateful appreciation to those organizations which have permitted the use of material from their publications, especially the Soil Conservation Service, United States Department of Agriculture, whose material was used in appendix A; and the Corps of Engineers, United States Department of the Army, whose Technical Manual TM 5–545 was freely used in the preparation of part D of chapter IV. Acknowledgments to other organizations furnishing a lesser amount of material are given throughout the text.

Floyd E. Dominy

FLOYD E. DOMINY,
Commissioner.

Contents

Chapter II—Flood Studies—Continued

B. PROCEDURES—Continued

C. SELECTED BIBLIOGRAPHY

Chapter III—Selection of Type of Dam

A. CLASSIFICATION OF TYPES

B. PHYSICAL FACTORS GOVERNING SELECTION OF TYPE

Chapter III—Selection of Type of Dam—Con.

C. LEGAL, ECONOMIC, AND ESTHETIC CONSIDERATIONS

Chapter IV—Foundations and Construction Materials

A. SCOPE OF INVESTIGATIONS

B. SOURCES OF INFORMATION

C. SOIL CLASSIFICATION

Chapter IV—Foundations and Construction Materials—Continued

D. ROCK CLASSIFICATION

E. SURFACE EXPLORATIONS

F. SUBSURFACE EXPLORATORY METHODS

Chapter IV—Foundations and Construction Materials—Continued

G. SAMPLING

H. LOGGING OF EXPLORATIONS

I. FIELD AND LABORATORY TESTS

J. BIBLIOGRAPHY

Chapter V—Earthfill Dams

A. INTRODUCTION

B. DESIGN PRINCIPLES

C. FOUNDATION DESIGN

Chapter V—Earthfill Dams—Continued

C. FOUNDATION DESIGN—Continued

D. EMBANKMENTS

E. EMBANKMENT DETAILS

F. DESIGN EXAMPLES OF SMALL EARTHFILL DAMS

Chapter V—Earthfill Dams—Continued

F. DESIGN EXAMPLES OF SMALL EARTHFILL DAMS—Continued

G. BIBLIOGRAPHY

Chapter VI—Rockfill Dams

A. GENERAL

B. FOUNDATION DESIGN

C. EMBANKMENT DESIGN

Chapter VI—Rockfill Dams—Continued

D. DESIGN OF UPSTREAM FACING

E. BIBLIOGRAPHY

Chapter VII—Concrete Gravity Dams

A. INTRODUCTION

B. FORCES ACTING ON THE DAM

C. REQUIREMENTS FOR STABILITY

D. DAMS ON PERVIOUS FOUNDATIONS

Chapter VII—Concrete Gravity Dams—Con.

E. DETAILS OF LAYOUT AND DESIGN

F. BIBLIOGRAPHY

Chapter VIII—Spillways

A. GENERAL

B. DESCRIPTION OF SERVICE SPILLWAYS

Chapter VIII—Spillways—Continued

B. DESCRIPTION OF SERVICE SPILLWAYS—Continued

C. HYDRAULICS OF CONTROL STRUCTURES

D. HYDRAULICS OF FREE-FLOW DISCHARGE CHANNELS

Chapter VIII—Spillways—Continued

E. HYDRAULICS OF TERMINAL STRUCTURES

F. HYDRAULICS OF SPILLWAYS

Chapter VIII—Spillways—Continued

G. STRUCTURAL DESIGN DETAILS

H. BIBLIOGRAPHY

Chapter IX—Outlet Works

A. GENERAL

B. OUTLET WORKS COMPONENTS

Chapter IX—Outlet Works—Continued

C. HYDRAULIC DESIGN

D. STRUCTURAL DESIGN DETAILS

E. BIBLIOGRAPHY

Chapter X—Diversion during Construction

A. DIVERSION REQUIREMENTS

Chapter X—Diversion during Construction—Continued

B. METHODS OF DIVERSION

C. SPECIFICATIONS REQUIREMENTS

D. BIBLIOGRAPHY

Chapter XI—Maintenance and Operation

A. GENERAL

B. INSPECTION AND MAINTENANCE OF DAMS

C. OPERATION

Appendix A—Estimating Rainfall Runoff from Soil and Cover Data

Appendix B—Hydraulic Computations

A. HYDRAULIC FORMULAS

Appendix B—Hydraulic Computations—Con.

A. HYDRAULIC FORMULAS—Continued

B. FLOW IN NATURAL CHANNELS

C. BIBLIOGRAPHY

Appendix C—Structural Design Data

Appendix D—Soil Mechanics Nomenclature

Appendix E—Construction of Embankments

Appendix F—Concrete in Construction

A. CONCRETE AND CONCRETING MATERIALS

B. DESIGN OF CONCRETE MIXES

Appendix F—Concrete in Construction—Con.

C. MANUFACTURE, PLACEMENT, CURING, AND INSPECTION OF CONCRETE

D. BIBLIOGRAPHY

Appendix G—Sample Specifications

A. EXCAVATION

Appendix G—Sample Specifications—Con.

A. EXCAVATION—Continued

B. EMBANKMENT

Appendix G—Sample Specifications—Con.

B. EMBANKMENT—Continued

C. PRESSURE GROUTING

D. CONCRETE SPECIFICATIONS

Appendix G—Sample Specifications—Con.

D. CONCRETE SPECIFICATIONS—Continued

Appendix G—Sample Specifications—Con.

D. CONCRETE SPECIFICATIONS—Continued

E. MISCELLANEOUS

Appendix G—Sample Specifications—Con.

E. MISCELLANEOUS—Continued

F. BIBLIOGRAPHY

LIST OF TABLES

LIST OF TABLES IN APPENDICES

LIST OF FIGURES

LIST OF FIGURES IN APPENDICES

Project Planning

A. F. JOHNSON [1]

A. PURPOSE OF THE DEVELOPMENT

1. *General.*—In this text, the term "project" means a water resource development. It may be small or large, simple or complex, serving one purpose or several, but it should provide the facilities to accomplish the optimum development of related physical resources. Although this text is concerned with small dams as one type of facility, the investigations and studies made for these dams must be considered in relation to the function they perform in accomplishing the purposes of the project as a whole. The project's objectives, purposes, and scope determine what must be investigated with respect to dams.

In many cases, the project will be of a dual or multipurpose type. For this reason, the investigations may encompass a large number of matters, some or all of which will influence the selection of a dam site, the size of the dam, and the purposes it serves. Hence, the entire project must be investigated as a unit before the design requirements for a single feature, such as a dam, can be firmly established. Each project purpose and each increment of its size or scope must justify inclusion in the project by some appropriate measure of feasibility or justification which is usually related to the benefits it produces, the need it serves, or the investment it can repay with or without interest.

Feasibility studies of dams and reservoirs should always consider possible objections from a public health and nuisance standpoint, and a proper effort should be made to mitigate the damages involved. The exposed bed of a reservoir, as it is drawn down, is not only unattractive but may make access to the water difficult. As deposits of sediment dry, odors from decaying vegetation or windblown dust may produce a nuisance and actual damage to health and property. In some cases, sewage detention may augment the hazard. Impounded fresh water held at a constant level makes an ideal breeding place for mosquitoes, thereby creating a nuisance and the possibility of transmitting malaria and encephalitis.

Many reservoirs for which this text is intended will be in regions affected by drought and subject to flash floods. Under these climatic conditions, floodwater erosion of the watershed and streambanks will fill the streams with sediment to be caught in the reservoirs. The accumulations of sediment may soon reduce the usefulness of a reservoir and ultimately will completely eliminate its capacity. Loss of capacity and other damages due to silting of reservoirs and changes of the regimen of silt-laden streams as a result of reservoir operation, should be considered for all proposed projects. The remaining paragraphs of this part of the chapter present pertinent aspects of common purposes, with particular emphasis upon design requirements for dams and reservoirs.

2. *Irrigation.*—The supply of water must be adequate for successful irrigation (considering occasional tolerable shortages) at an economically reasonable cost per unit of area, both for capital investment and for operation, maintenance, and replacement. The quality of the water must be such that it will not be harmful to the crops or to the soils on which it will be used. If the distribution system is to depend on gravity flow, the reservoir must be high enough above the irrigated area to provide the necessary head for adequate delivery.

3. *Domestic and Municipal Purposes.*—The supply of water must be adequate to serve the requirements. The present demand and a surplus to

[1] Engineer, Field Investigations Branch, Bureau of Reclamation.

care for the reasonably predictable increases in requirements are important considerations.

The quality of the water must be such that it can be rendered potable and usable for domestic and most industrial purposes by economical treatment methods. It should meet State public health standards as to bacterial purity. Standards with regard to taste, color, odor, and hardness may vary with different sections of the country. The degree to which objectionable characteristics can be corrected will depend on their nature and concentration in the raw supply, and on the cost of necessary remedial measures.

Control and protection of small watershed areas is desirable for municipal water supply reservoirs. Even though the entire watershed cannot be purchased or otherwise provided protection from contamination, an effort should be made to provide control through agreements or purchase of shoreline lands.

4. *Industrial Use.*—Although quality of water for municipal services is ordinarily satisfactory for industrial uses, certain industrial processes require more exacting standards with respect to freedom from chemicals injurious to equipment or to the product manufactured.

5. *Stock Water.*—The quality of the water for stock consumption must be suitable for the purpose. The pond should be situated where it is accessible to the stock, either directly or by the economical use of ditches or pipes.

6. *Power Development.*—Where power generation is included, the capacity of the power-generating equipment and the load demand are closely related to the quantity of water available and the amount of storage provided. The height of a power dam is usually dictated by these requirements. Special studies of this nature are outside the scope of this text.

7. *Flood Control.*—In the study and design of flood control projects and structures, the following factors should be considered:

(1) The relation of the cost of control to the benefits to be derived through the reduction of cumulative damage, should be favorable as compared to alternative means of obtaining similar benefits, and in the light of public interest.

(2) The temporary storage must be sufficient to lower the major peak flows or to decrease the frequency of minor floods.

(3) So far as is practicable, the method of control should be automatic rather than manual.

(4) Any flood control must be effective. An implied downstream safety that does not exist is more dangerous than having no control at all.

8. *Recreation.*—The following factors should be considered in regard to development of recreational projects:

(1) There must be an adequate supply of water to provide for evaporation losses and to maintain the water level within those limitations assumed as a basis for the recreational and residential development of the shoreline.

(2) The water must be free of pollution within practical limits.

(3) If bathing is one of the purposes, there must be an adequate depth of water in the vicinity of a gently sloping shore.

(4) Where there is considerable recreational use of varied nature, the shoreline should be zoned to preserve privacy and to provide separation of conflicting uses, such as residential, camping, day picnicking, bathing, or boating. Minimum facilities for public use and safety, such as access roads, parking, boat ramps or docks, fireplaces and tables, and sanitary facilities, are commonly provided. Other facilities, such as motels, stores, and commercial amusements, are generally provided under concession.

(5) The shoreline should have a relatively steep gradient where possible, so that a slight lowering of the water surface will expose a minimum of area. Also, the normal range of the operating level should not include extensive areas of flat shoreline which will be unsightly when uncovered. Probable use of lands on shorelines should be considered in proposed plans and in estimates of required reservoir rights-of-way. Easements for maximum floods and floods of rare frequency are less costly than outright purchase and permit private use of the shorelines.

9. *Wildlife.*—Projects to impound water for wildlife purposes should not be promoted without the advice of a biologist. An ability to catch fish or shoot game birds is not a guarantee of knowledge required for raising them. The following factors must be considered:

(1) There must be sufficient depth and supply of water to maintain livable conditions for wildlife throughout the dry season.

(2) Extensive fluctuations in water level are inimical to fish and other wildlife, as they prevent

or destroy the development of aquatic vegetation required for food.

(3) Satisfactory quality of the water must be assured. Pollution may poison waterfowl or fish and reduce the oxygen content to the point where many game fish cannot survive. Excessively acid or highly alkaline water is harmful to many kinds of wildlife.

(4) The water and the basin must be suited to the production of the right kind of food supply and must provide adequate shelter and freedom from disturbance by man.

10. *Water Storage for Streamflow Regulation.*—There is a need for projects of this type in those regions where streamflow either ceases entirely or is reduced to extremely low values during parts of the year. Where such natural streamflow is the principal source of water supply for one or more communities and where a dependable streamflow is necessary for the dilution of waste after proper economical treatment, water storage for streamflow regulation may be justified. For such projects, it must be determined that:

(1) The dependable streamflow, when properly regulated, is sufficient to produce the minimum values of regulated flow required for the purpose, after expected losses (including evaporation) have been deducted; and

(2) Storage for this purpose will not result in an objectionable alteration of the quality of the water.

11. *Miscellaneous Water Conservation Projects.*—Occasionally projects are proposed to regulate the water level in shallow lakes, swamps, or bogs for purposes other than those heretofore enumerated. This classification also includes projects for the detention or diversion of streamflow to conserve it by transforming surface water to ground water through the process of infiltration.

It should be noted that natural shallow lakes, swamps, and bogs generally exist because of an underlying impervious subsoil and that additional storage for surface water at such locations will rarely be effective in increasing ground water, unless the stored water can be conveyed to and flooded over other areas where infiltration can take place.

Projects ot the type under discussion will generally result in large increases in the evaporation and transpiration losses because of the enlargement of water surface, the increased time and exposure of the surface water to evaporation, or possibly an increase in the area of semiaquatic plants such as sedge grass and reeds. The net results on such projects, after deducting such losses, must be substantially beneficial. In many cases, there is a danger that projects of this type will result in the loss of water through evaporation that might otherwise be beneficially used elsewhere as streamflow.

For projects involving water spreading or the detention of surface water to increase infiltration and percolation opportunities, it must be determined that the soil characteristics will permit infiltration to occur in a quantity which justifies the project economically. The Groundwater Branch of the Geological Survey (Department of the Interior), the Bureau of Land Management (Department of the Interior), the Soil Conservation Service (Department of Agriculture), or appropriate State bureaus should be consulted with regard to projects of this nature.

B. PROJECT STUDIES

12. *Tests of Feasibility.*—The objective in project planning is the determination of the projects' feasibility. This involves studies which will permit a sound analysis and conclusion with respect to the specific engineering-economic considerations. These are primarily:

(1) That the project is responsive to an urgent present or anticipated social or economic need;

(2) That the project as planned will adequately serve the intended purpose; and

(3) That the services proposed to be performed through the project and the benefits it will produce will justify the cost.

The study should determine that the difficulties inherent in facility sites which affect economy, safety of construction, and quality of operation have been satisfactorily foreseen; and that the designs are technically sound and reasonably representative of the actual structures that may be expected to be built after more detailed investigation. The soundness of the conclusions regarding these matters will depend to a considerable

degree on the completeness and accuracy of the investigation.

13. *Esthetic Value.*—Esthetic values may be of great importance to a project. In the location and design of dams and other major structures, they should be given appropriate consideration which should be recognized in the first studies made and carried out in later studies and in construction operations. However, esthetic values should not be allowed to outweigh the safety or adequacy of the structural design.

The application of the principles involved varies so greatly that the solution with respect to any particular dam requires individual study. This subject cannot be properly covered within the scope of this text. When esthetic considerations are important, it is recommended that a qualified landscape architect be consulted; where warranted, an architect should collaborate in making the design.

14. *Extent of Studies.*—There is no simple rule for determining the extent of the investigation which is necessary in any particular case. For example, a dam that is to be constructed on a pervious or a weak foundation to impound a depth of water of 15 feet will require much more foundation investigation than a 50-foot-high dam to be built on solid, unfractured rock under a shallow soil mantle. Within the limits of height treated in this text, it may be said that the size of the physical structure has little relation to the extent of the investigation necessary. The maximum justifiable investigation cost is, however, limited by the magnitude of the project. The project is generally unjustified if the cost of the necessary investigation would offset a large portion of the constructed project's value. A cost reduction accomplished by the elimination of a portion of the fundamental investigation is rarely a saving. It generally results in unanticipated construction or functional costs.

15. *Stages of Investigation.*—Investigation, if carried to completion, is an expensive and time-consuming phase of project development. Moreover, it may indicate that the project is not economically or technically sound. Hence, an investigation should be planned and executed so that the probable soundness of the project will be determined as early and as inexpensively as possible. To accomplish this objective, the investigation may be divided into as many as three stages. The first, or reconnaissance stage, is designed primarily to support a decision on whether to proceed with more detailed investigations on the basis of rough data and shortcut studies. The second, or feasibility stage, determines the scope, magnitude, essential plan and feature, and the approximate benefits and costs of the project with sufficient dependability to support project authorization or approval for construction. The third, the specifications stage, supplements the feasibility stage to the degree needed for preparation of final plans and specifications after authorization or approval has been obtained and construction is imminent.

Many of the smaller projects will not require, at the specifications stage, any information in addition to that already obtained in the feasibility studies. The larger and more difficult projects will often require extensive additional surveys and investigations. However, project size is not the sole criterion with respect to the necessity for further detailed studies. This may rest on a question of complexity of the site, of the foundation conditions, and often of the hydrological factors.

The following sections discuss the elements of project studies within the scope of the various stages of investigation.

16. *Related Projects and Studies of Need.*—The proposed project should be consistent with any long-range planning program which may have been adopted for the vicinity. The entire area to be served by the proposed project should be studied to determine whether there will be conflicts with other projects of a similar nature for use of land or water resources, power drops, etc., or whether economies are possible by securing optimum development of resources through joint use.

If a potential project conflicts with other similar facilities, either completed or contemplated, it is advisable to refer the matter to appropriate State or local authorities, especially where future needs or opportunities must be preserved for the good of the locality, State, or Nation. The probable existing demands for the services to be rendered by the project as well as a reasonable estimate of future demand should be carefully determined.

17. *Development of the General Plan.*—The project plan normally originates with the desire to satisfy one sponsor's specific needs, objectives, or purposes. As support for project construction develops, those needs may increase, the objectives

may broaden, and the purposes may multiply through the processes of project formulation until selections of the final magnitude and scope are reached.

At the outset of the reconnaissance study, considerable basic data are usually available in the form of maps, aerial photographs, streamflow records, regional geological reports, census statistics, crop yields, market statistics, power loads, previous investigation reports, etc. The investigator must evaluate these data, supplement them with rough additional data, and conceive a workable basic plan that utilizes available resources to meet the needs. This basic plan may then be compared roughly with alternatives to accomplish the desired ends on progressively increased or diminished scope and scale. It will be possible, by judgment or cursory study, to eliminate many alternatives so that an approximate final plan emerges which includes purposes, approximate locations and heights of dams, and capacities of reservoirs, spillways, outlets, canals, powerplants, and other features. This plan will be refined in the feasibility and specification design stages. The character and scope of required feasibility surveys, investigations, and studies may be visualized by judgment and experience from a consideration of the plan and knowledge of its components acquired during the reconnaissance stage study.

Governmental agencies have obtained a considerable amount of basic data of the nature mentioned above for many areas. The larger public libraries are generally designated as public document depositories and maintain more or less complete files of all United States Government publications and, of course, many State and local publications as well. U.S. Government publications in stock may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D.C., and extensive lists of publications are available from that office and from issuing agencies. Individual agencies commonly provide sales rooms at their principal offices. Sponsors and engineers for small projects should consult with State and county planning officials, local offices of the Federal data-collecting agencies, and public libraries concerning the availability of pertinent data. Following is a source list of possible data:

484556 O—60——3

(a) *Topographic Maps:*
   (1) Quadrangle maps—U.S. Department of the Interior, Geological Survey, Topographic Division; and U.S. Department of the Army, Army Map Service.
   (2) River plans and profiles—U.S. Department of the Interior, Geological Survey, Conservation Division.
   (3) National parks and monuments—U.S. Department of the Interior, National Park Service.
   (4) Federal reclamation project maps—U.S. Department of the Interior, Bureau of Reclamation.
   (5) Local areas—commercial aerial mapping firms.

   The availability of topographic maps is discussed in chapter IV.

(b) *Planimetric Maps:*
   (1) Plats of public land surveys—U.S. Department of the Interior, Bureau of Land Management.
   (2) National forest maps—U.S. Department of Agriculture, Forest Service.
   (3) County maps—county surveyor or county engineer.
   (4) City plats—city or county recorder.
   (5) Federal reclamation project maps—U.S. Department of the Interior, Bureau of Reclamation.

(c) *Aerial Photographs:*
   (1) The following agencies have aerial photographs of portions of the United States: U.S. Department of the Interior, Geological Survey, Topographic Division; U.S. Department of Agriculture, Commodity Stabilization Service, Soil Conservation Service, and Forest Service; U.S. Department of Commerce, Coast and Geodetic Survey; U.S. Air Force; various State agencies; commercial aerial survey and mapping firms.

   The availability of aerial photographs is discussed in chapter IV.

(d) *Transportation Maps:*
   (1) State transportation maps—U.S. Department of Commerce, Bureau of Public Roads.
   (2) State and county highway maps—State highway departments.

(3) Sectional aeronautical charts—U.S. Department of Commerce, Coast and Geodetic Survey.

(e) *Triangulation and Benchmarks:*

(1) U.S. Department of Commerce, Coast and Geodetic Survey; and U.S. Department of the Interior, Geological Survey, Topographic Division.

(f) *Geology:*

(1) Geologic maps and reports—U.S. Department of the Interior, Geological Survey, Geologic Division; and State geological surveys or departments.

(*Note.*—Some regular quadrangle maps show geological data also.)

Geologic maps are discussed in chapter IV.

(g) *Soils:*

(1) County soil survey reports—U.S. Department of Agriculture, Soil Conservation Service.

(2) Land use capability surveys—U.S. Department of Agriculture, Soil Conservation Service.

(3) Land classification reports—U.S. Department of the Interior, Bureau of Reclamation.

Agricultural soil maps are discussed in chapter IV.

(h) *Climatological Data:*

(1) Daily weather reports—U.S. Department of Commerce, Weather Bureau.

(2) Climatological data (monthly and annual summaries); includes precipitation, temperature, evaporation, and wind velocity data—U.S. Department of Commerce, Weather Bureau.

(3) Daily synoptic weather maps—U.S. Department of Commerce, Weather Bureau.

(4) Hydrologic bulletin—U.S. Department of Commerce, Weather Bureau.

(5) Technical papers—U.S. Department of Commerce, Weather Bureau.

(6) Hydrometeorological reports—U.S. Department of Commerce, Weather Bureau; and U.S. Department of the Army, Corps of Engineers.

(7) Cooperative study reports—U.S. Department of Commerce, Weather Bureau; and U.S. Department of the Interior, Bureau of Reclamation.

(8) Unofficial observers—such as banks, local businessmen, farmers, etc.

(i) *Streamflow Data:*

(1) Water supply papers—U.S. Department of the Interior, Geological Survey, Water Resources Division.

(2) Reports of State engineers.

(3) Annual reports—International Boundary and Water Commission, United States and Mexico.

(4) Annual reports—various interstate compact commissions.

(5) Water right filings, permits—State engineers, county recorders.

(6) Water right decrees—district courts.

(j) *Sedimentation:*

(1) Water supply papers—U.S. Department of the Interior, Geological Survey, Quality of Water Branch.

(2) Reports—U.S. Department of the Interior, Bureau of Reclamation; and U.S. Department of Agriculture, Soil Conservation Service.

(k) *Quality of Water:*

(1) Water supply papers—U.S. Department of the Interior, Geological Survey, Quality of Water Branch.

(2) Reports—U.S. Department of Health, Education, and Welfare, Public Health Service.

(3) Reports—State public health departments.

(l) *Irrigation and Drainage Data:*

(1) Agricultural census reports—U.S. Department of Commerce, Bureau of the Census.

(2) Agricultural statistics—U.S. Department of Agriculture, Agricultural Marketing Service.

(3) Annual crop summaries on Federal reclamation projects—U.S. Department of the Interior, Bureau of Reclamation.

(4) Reports of State departments of agriculture.

(m) *Power Data:*

(1) Directory of Electric Utilities—McGraw-Hill Publishing Co.

(2) Directory of Electric and Gas Utilities

in the United States—Federal Power Commission.

(3) Power rates—power companies, public utility districts, rural electric cooperatives, etc.

(4) Power markets—State universities, Bureau of Business Research.

(5) Reports—various power companies, public utilities, State power commissions, etc.

(n) *Bulletins:*

(1) Interagency Committee on Water Resources.

(2) Annotated bibliographies on hydrology.

(3) Annotated bibliographies on sedimentation.

(4) Population census—U.S. Department of Commerce, Bureau of the Census.

(o) *Basin and Project Reports and Special Reports:*

(1) U.S. Department of the Army, Corps of Engineers.

(2) U.S. Department of the Interior, Bureau of Land Management, Bureau of Mines, Bureau of Reclamation, Fish and Wildlife Service, and National Park Service.

(3) U.S. Department of Agriculture, Soil Conservation Service.

(4) U.S. Department of Health, Education, and Welfare, Public Health Service.

(5) State departments of water resources, departments of public works, power authorities, and planning commissions.

(p) *Consultation:*

(1) Sanitation and public health—U.S. Department of Health, Education, and Welfare, Public Health Service; State departments of public health.

(2) Fish and wildlife problems—U.S. Department of the Interior, Fish and Wildlife Service; State game and fish departments.

(3) Municipal and industrial water supplies—city water departments; State universities; Bureau of Business Research; State water conservation boards or State public works departments.

(4) Watershed management—U.S. Department of Agriculture, Soil Conservation Service, Forest Service; U.S. Department of the Interior, Bureau of Land Management, Bureau of Indian Affairs.

(q) *Designs and Specifications, Water Rights.*—Consult State engineer.

18. *Outline of Investigations.*—The outline given below provides a guide for the field engineer by indicating the items which should be considered for the investigations of dams and reservoirs. The outline includes items many of which may not be applicable in a specific instance. Most of the items in the outline are discussed in detail in the remainder of this chapter.

I. *General Data Required:*

A. Location and vicinity map:

1. Location of project.
2. Location of existing works affected by proposed development.
3. Existing location of highways, railroads, and other public utilities and proposed relocations.
4. Location of proposed construction headquarters, camp, or town, access roads, airports, etc.
5. Railroad shipping points.
6. Stream gaging and sampling stations, meteorological stations, etc.

B. Hydrologic data:

1. Streamflow records, including daily discharges, monthly volumes, and momentary peaks.
2. Streamflow and reservoir yield.
3. Project water requirements, including allowances for irrigation and power efficiencies, conveyance losses, reuse of return flows, and stream releases for fish; and dead storage requirements for power, recreation, fish and wildlife, etc.
4. Flood studies, including inflow design floods and floods to be expected during periods of construction.
5. Sedimentation and quality studies, including sediment measurements, analysis of dissolved solids, etc.
6. Data on ground-water tables in the vicinity of the reservoir and damsite.
7. Water rights, including interstate compacts and international treaty effects, and contractual agreements with local districts, power companies, and individuals for subordination of rights, etc.

C. Climatic data:
   1. Monthly temperatures and rainfall and storm intensities.
   2. Evaporation rates.
   3. Maximum, minimum, and mean temperatures.
   4. Wind directions and velocities.
   5. Ice thickness.

D. Geologic data:
   1. Geological report by qualified geologist.
   2. Discussion of geologic formations, particularly such as cavernous limestone or other soluble formations, exposed lava, exposed gravel, and glacial deposits of a permeable nature that might contribute to serious reservoir leakage.
   3. Ground-water table observations.
   4. Presence of deleterious mineral and salt deposits.
   5. Photographs showing basin and character of lands.
   6. Geologic cross sections where necessary.

II. *Reservoir Data:*

A. Reservoir map:
   1. Topography.
   2. Horizontal and vertical controls, preferably a triangulation survey system.
   3. Coordinate system.
   4. Cultural classification of reservoir lands, land ownership, and status in connection with acquisition of rights-of-way, easements, etc.
   5. Ownership boundaries and names of owners.

B. Road and public utility surveys:
   1. Relocation and reconstruction of railroads and highways.
   2. Relocation and reconstruction of public utilities.
   3. Preliminary report of joint reconnaissance made with the municipality or owner of public utilities with approximate costs of relocation, including the necessity for location surveys and who will perform the construction.

C. Miscellaneous data:
   1. Estimate of probable life of reservoir—that is, considering loss of capacity due to silting.
   2. Land evaluation surveys, including tabulations of areas and estimated costs for purchase and easements; discussion of necessity for appraisal survey and discussion of possibility of securing easements for submergence of lands during maximum and infrequent floods.
   3. Tabulation of areas to be cleared, with estimated cost.
   4. Description of buildings, fences, and farm improvements which must be removed or salvaged, with estimates of cost.
   5. Description of lands adjacent to the proposed reservoir for public use, recreation, or other purposes.
   6. Economic or physical limitations to maximum reservoir flow line.
   7. Discussion of limitations to reservoir fluctuations.

III. *Data for Dams:*

A. Damsite map:
   1. Topography of damsite and dike areas.
   2. Horizontal and vertical controls, preferably by a triangulation system.
   3. Coordinate system grid.
   4. Location of rock outcrops and apparent geological features.
   5. Location of manmade improvements in existing works at the site.
   6. Location of drill holes, test pits, and other foundation explorations.

B. Foundation explorations:
   1. Sufficient drill holes, auger holes, and/or test pits to determine character and depth of overburden for feasibility and specifications designs.
   2. Description and logs of exploration, including ground elevation at the holes, location coordinates, and sufficiently detailed remarks for a clear interpretation of records.
   3. Samples.
   4. Sufficient explorations to determine character of bedrock or impervious foundation strata for feasibility and specifications designs.

C. Materials exploration:
   1. Location and description of character of proposed material to be used in the construction of the dam, including earth, sand-gravel for aggregate and

embankment, and rock for rockfill and riprap.

2. Map of borrow areas, showing location of test holes made for feasibility and specifications designs.
3. Logs of explorations.
4. Representative samples of materials in borrow areas.

D. Tailwater data:

1. Stage-discharge curves for streams, if available.
2. Cross sections of streams, with dated water surface elevations above and below dam.
3. Tailwater and backwater curves for stated high-water marks.

E. Local conditions controlling design of the dam:

1. Requirements for roadways.
2. Requirements for fishways or fish conservation measures.
3. Requirements for replacement or provisions for existing works.
4. Requirement of permanent building or quarters for operators.
5. Effect of local conditions as regards spillway and outlet gates, especially winter conditions, etc.
6. Availability of electric power for operation of mechanical equipment.
7. Capacities and elevations of required outlets as determined by local conditions.

F. Local conditions affecting construction:

1. Additional transportation facilities required for construction, including access roads, etc.
2. Location surveys for railroads, highways, or airports.
3. Requirement for improvements to existing transportation facilities.
4. Estimated cost for sufficient data for preparation of estimates for transportation facilities.
5. Hauling distance from nearest railroad shipping point and local trucking rates.
6. Availability of electric power for construction purposes.
7. Requirements for construction camp, with estimated population and quarters required for supervisory and construction employees; required water supply and sanitation facilities; local laws regarding sanitation, stream pollution, etc.

19. *Planning Programs for Surveys and Investigations.* During the reconnaissance study, it is desirable to consider plans for detailed surveys and investigations, particularly if it is known that a feasibility study probably will be made. In planning such a program, consideration should be given to the following items: (1) The personnel required, (2) housing and subsistence for personnel, (3) location of field office if required, (4) transportation and other equipment, material, and supplies required for this work, (5) character of additional dam foundation investigation and equipment required, (6) availability of local labor and equipment, (7) arrangements with private landowners for entry to sites to avoid trespass of lands during surveys and explorations, (8) transportation of drilling equipment to isolated and inaccessible locations, (9) consideration of horizontal and vertical controls for surveys and available data from other surveys such as railroad and highway surveys, county engineers' surveys, etc., (10) location of stream gaging stations and available hydrologic data, (11) climatic conditions for work, (12) sanitary conditions if pollution will be a factor limiting the utilization of the project, and (13) estimates of time and funds required for work.

In the case of public works, investigation and construction funds generally must be budgeted and appropriated, and the time necessary for these processes must be anticipated. With local and private developments, similar time requirements usually are involved in public elections, bond issues, or other means of financing.

20. *Mapping.*—The project location should be shown on a general map, using a base of appropriate scale. U.S. Geological Survey quadrangle maps, particularly the 7½-minute quadrangles, 1 inch equal to 2,000 feet, are appropriate for preliminary purposes in locating land areas and principal features and in determining approximate reservoir capacities. If no topographic maps are available, rough reconnaissance maps, defined by a minimum of controlling points or sketched from cross sections, can be used for comparative studies. The general map should show governing elevations of water courses, canal routes, dams, and important cultural and occupational features such

as woods, cultivated lands, swamps, roads, and railroads. Features pertinent to reservoirs and dams should be shown on separate maps of the reservoir and of the damsite.

Profiles at damsites may serve for reconnaissance purposes. During the feasibility stage, if practicable, and certainly during specifications investigations, reliable topographic maps of irrigable areas, reservoirs, dams, conduit locations, and other structural sites must be obtained. These should be to a convenient scale and contour interval laid out on a coordinate grid, preferably tied into State coordinate systems and public lands survey systems, with horizontal and vertical controls based on U.S. Coast and Geodetic Survey or U.S. Geological Survey nets and datum. Control points should be permanently monumented.

Topographic map scales should range from 1 inch equals 400 feet for small reservoirs to 1 inch equals 1,000 feet or 2,000 feet for very large reservoirs, with contour intervals ranging from 2 feet for small reservoirs created by low dams to 5 feet for reservoirs of greater depth. Damsite maps should be in greater detail—1 inch equals 20 feet for sites for the smallest dams, up to 1 inch equals 100 feet or 200 feet for sites in flat terrain, with contour intervals of 2 or 5 feet depending on the steepness of the topography. Coverage should extend beyond the limits of the reservoir or dam to include rights-of-way, utility relocations, camp and service areas, nearby borrow areas, etc. River sections should be obtained from the dam axis to 2,000 feet or more downstream as required for tailwater curves, degradation studies, channel changes, etc. Topography should be obtained at the heads of reservoirs where sediment deltas or backwater levels will be important. To an increasing degree, topographic surveys are utilizing photogrammetric methods and much of the work formerly done by project survey parties is done under contract by firms which specialize in photogrammetric surveys. Property boundaries and ownerships are desirable where rights-of-way must be purchased.

The location and elevations of all drill holes, test pits, trenches, etc., and of control monuments and temporary points should be shown on the detailed topographic maps of the damsite.

The principal difference between the surveys and maps required for feasibility stage investigations and for specifications design is that, for the latter, certain features must be more complete and accurate so as to provide all of the information needed for the actual design of the structures. In many cases, an evaluation of the factors entering into the justification of the project may have required that surveys and investigations be carried out in considerable detail during the feasibility studies. If the feasibility stage surveys and maps are up to acceptable standards of accuracy, it will only be necessary to bring them up to date to reflect cultural changes and to add detailed data, such as supplemental drill holes. Where the feasibility studies have been based on reconnaissance surveys and sketch maps, extensive field surveys must be carried out in connection with the specifications stage investigations. Substandard surveys and maps should be brought up to standard or be replaced by new ones which have sufficient accuracy and detail for final design purposes.

21. *Hydrologic Investigations.*—Hydrologic investigations which may be required for project studies include the determination of the following: Yield of streamflow, reservoir yield, water requirements for project purposes, sediment which will be deposited in the reservoir, flood flows, and groundwater conditions.

The most accurate estimate possible must be prepared of the portion of the streamflow yield that is surplus to senior water rights as the basis of the justifiable storage. Reservoir storage will supplement natural yield of streamflow during low-water periods. Safe reservoir yield will be the quantity of water which can be delivered on a firm basis through a critical low-water period with a given reservoir capacity. Reservoir capacities and safe reservoir yields may be prepared from mass curves of natural streamflow yield as related to fixed water demands or from detailed reservoir operation studies, depending upon the study detail which is justified. Reservoir evaporation and other incidental losses should be accounted for before computation of net reservoir yields. The critical low-water period may be 1 drought year or a series of dry years during the period of recorded water history. Water shortages should not be contemplated when considering municipal and industrial water use. For other uses, such as irrigation, it is usually permissible to assume tolerable water shortages during infrequent drought periods and thereby increase water use during normal periods with consequent greater project

development. Tolerable irrigation water shortages will depend upon local conditions and crops to be irrigated. If the problem is complex, the consulting advice of an experienced hydrologist should be secured.

The annual rate at which sediment will be deposited in the reservoir should be ascertained to insure that sufficient sediment storage is provided in the reservoir so that the useful functions of the reservoir will not be impaired by sediment deposition within the useful life of the project or the period of economic analysis, say 50 to 100 years. The expected elevation of the sediment deposition may also influence the design of the outlet works, necessitating a type of design which will permit raising the intake of the outlet works as the sediment is deposited. The method of determining the volume of sediment and the type of deposition to be expected will depend on the type of hydrologic data available. Preliminary estimates of this sediment volume can be based on assumed sediment yield rates expressed in acre-feet per square mile of drainage area. Selecting proper yield rates can generally be accomplished by: (1) Using a rate determined previously for a comparable drainage area, or (2) referring to various published reports [1, 2, 3, 4, 5, 6] [2] containing yield rate information for specific drainage basins.

Another valuable source of information on the sedimentation quantities carried by the rivers of the United States is the U.S. Geological Survey Water Supply Papers entitled "Quality of Water." These publications contain the records of suspended sediment loads at numerous measuring stations. They also supply data on the size analysis of river-borne sediments that are useful for other studies. A report by the Bureau of Reclamation [7] outlines a procedure for computing sediment yields using data contained in these water supply papers. Trap efficiency, defined as the percentage of total sediment load entrapped in the reservoir, also needs to be considered in the preliminary investigations. Reservoir trap efficiency depends on such factors as shape of reservoir basin, type of sediment material entering reservoir, method of reservoir operation, ratio of storage capacity and inflow, and age of reservoir. Brune [8] reports results of his studies on trap efficiencies which show close correlations between trap efficiency and the capacity-inflow ratio. His approach to the determination of trap efficiencies is among the best in practical application of current methods.

Another highly important factor in reservoir sedimentation is the derivation of the sediment distribution or deposition pattern within the reservoir. Generally, the same factors affecting trap efficiencies also have an influence on this pattern. Sediment deposition studies serve two major purposes: (1) To determine the sediment volume occupied within the various storage capacities allocated for specific uses such as flood control, irrigation, and dead storages, and (2) to determine the minimum elevation for the outlet sill. The Bureau of Reclamation has published a report [9] that contains a compilation of methods to compute the sediment deposition within a reservoir. A paper [10] published by the American Society of Civil Engineers also outlines computational procedures for sediment distribution studies.

Water requirements should be determined for all purposes contemplated in the project. For irrigation, consideration should be given to climatic conditions, soil types, type of crops, crop distribution, irrigation efficiency and conveyance losses, reuse of return flows, etc. For municipal and industrial water supplies, the anticipated growth of demand over the life of the project must be considered. For power, the factors to be considered are load requirements and anticipated load growth.

Project studies must include estimates of flood flows, as these are essential to the determination of the spillway capacity. Consideration should also be given to annual minimum and mean discharges and to the magnitudes of relatively common floods of frequencies up to about 10-year recurrence intervals, as this knowledge is essential for construction purposes such as diverting the stream, providing cofferdam protection, and scheduling operations. Methods of arriving at estimates of floodflows are discussed in chapter II. If the feasibility studies are relatively complete, the flood determination may be sufficient for design purposes. If, however, floodflows have been computed for purposes of the feasibility study without making full use of all available data, these studies should be carefully reviewed and extended in detail before the actual design of the structure is undertaken. Frequently, new data on storms,

---

[2] Numbers in brackets refer to items in the bibliography, sec. 32.

floods, and droughts become available between the time the feasibility studies are made and construction starts. Where such changes are significant, the flood studies should be revised and brought up to date.

Project studies should also include a ground-water study which may be limited largely to determining the effect of ground water on construction methods. However, some ground-water situations may have an important bearing on the choice of the type of dam to be constructed and on the estimates of the cost of foundations. Important ground-water information sometimes can be obtained in connection with subsurface investigations of foundation conditions.

As soon as a project appears to be feasible, steps should be taken in accordance with State water laws to initiate a project water right.

22. ***Investigations of Foundations and Materials.***—On all dam projects the watertightness of the reservoir, the suitability of the foundations for the dam and appurtenant structures, and the construction materials sources, are important geological and engineering considerations. The extent of investigations for resolving these problems should be commensurate with the importance of the project and with the stage of the investigation. Obtaining highly detailed information on subsurface conditions may exceed justifiable costs on most small projects; hence, reliance will have to be placed on judgment and experience rather than on a large amount of subsurface exploration and laboratory testing. For example, considerations of economy require a minimum of exploration for alternative dam sites prior to final selection. Once the site is established, however, the information to be obtained at each subsequent stage of the explorations should be of a quality suitable for use in specifications designs. Uneconomical duplication of effort and contractual difficulties are risked when incomplete data are obtained.

In the reconnaissance stage, sufficient information is obtained to select the damsite and to determine whether additional investigation is warranted. The first step is to search for and study all geological and soils data relating to the area, including maps, air photographs, and reports. Sources of such information are given in chapter IV. It may be difficult to apply the data from these sources directly to the design of the dam; however, the information that can be gleaned from them is of considerable value in planning field investigations and, later, in interpreting their results. The second step in the reconnaissance stage is a field examination of the site and surrounding area by the engineer who should be accompanied, if possible, by an engineering geologist. This examination should include construction materials sources and the reservoir and damsite geology that is visible on the ground.

It is important to record all of the data obtained in the reconnaissance stage. Even if economic factors at the time of the study preclude further investigations, changing economic conditions later may result in a reevaluation of the project; thus, costly duplication of earlier investigations can be avoided. Because reconnaissance data may be based primarily on limited surface information, large factors of safety should be used in estimates of potential quantities of construction materials, hauling distances, and depths for cutoffs.

The objective of the feasibility stage is to obtain data for a cost estimate. The estimate should be sufficiently accurate to determine whether the project is economically justified. The extent of the investigation depends on the data available from the reconnaissance stage. The accuracy of information required in the feasibility stage generally requires that subsurface explorations be performed. Detailed information on exploration methods, logging, and sampling procedures is given in chapter IV.

Additional boreholes may be needed in the specifications stage to answer critical questions posed by the feasibility designs, such as, what should be the depth of cutoff trenches, or if borrow material is scarce, what quantity is available. Earlier explorations may also have raised critical geological questions requiring resolution such as, will a fault zone encountered in one of the bore holes be of sufficient extent to create a design problem or is further exploration necessary to outline definitely a buried channel filled with highly pervious materials that intersects the dam foundations.

In this final investigation stage, a limited number of laboratory index tests should be made on representative samples. The purpose of these tests is to verify soil classifications and to obtain the moisture-density characteristics of proposed borrow area soils. Permeability tests rarely will be required if adequate soil classifications are made.

However, very sandy soils may be tested if the design requires specific permeability information. The properties of proposed sources of rock for riprap and rockfill and of concrete aggregate should be investigated as described in chapter IV.

23. *Sanitary Studies.*—The necessity for sanitary study is determined by the degree to which pollution will be a factor in limiting the utilization of the proposed project. All possible sources of pollution from human, animal, and industrial waste should be investigated and evaluated. If municipalities are situated on the catchment basin, their sewage disposal systems must be investigated. Water samples adequately covering the period of the year during which the proposed project will be in use should be taken from the water course below the municipality and analyzed, particularly if there is a sewer outfall emptying into the stream. If the disposal system includes a bypass to the stream around the treatment plant, its effect should be evaluated and definite provisions made whereby the authorities in charge of the reservoir project will be notified prior to its use. Often a municipality has an inadequate disposal system but has plans for eventual improvement. In such cases, it may be necessary to plan the proposed project for limited use, pending the correction.

24. *Recreation and Fish and Wildlife.*—These investigations, especially those relating to fish and wildlife, require the services of experts in the field as discussed in sections 8 and 9. Recreational investigations include estimates of the number of people that might visit the development one or more times in a season, the probable length of visit, the possible uses and need for zoning the area, and a comparison of the area with competing areas.

Fish and wildlife may be affected beneficially or detrimentally. The magnitude of the impact should be estimated, and plans should be included for mitigating damage to the extent warranted through provision of dead storage in reservoirs, regulated releases, fish ladders to facilitate migratory movement, fish screens to prevent access, provision of vegetative cover for nesting areas and for sanctuaries, provision of areas for production of feed, etc.

25. *Design of Structures.*—Technical questions involved in the design of small dams of various types and in the designs and layout of the appurtenant structures, such as outlet works and spillways, are presented in the various chapters covering outlets, spillways, and different types of dams.

Except for very minor structures, most States require submittal of plans for review and inspection in the case of non-Federal works. Engineers or owners considering dam construction in any State should obtain concise information on what type of control is exercised by the State, as well as the name and address of the appropriate official and agency which exercises such control. Permits or licenses must be secured from the Federal Power Commission for power projects, and approval must be obtained from the Department of Defense if structures on navigable streams are involved.

Before proceeding with design of a dam, the designer should consult with the appropriate officials to determine the requirements of the supervising agency.

26. *Preparation of Cost Estimates.*—Rough overall estimates of project feature costs are commonly made during the reconnaissance stage for the purpose of comparing alternative sites and for determining the size and scope of development. More detailed estimates involving quantities and unit costs are necessary for inclusion in feasibility reports supporting authorization or approval for construction, after plan formulation studies have established the optimum scale for economic soundness of the development. Estimates for dams and reservoirs should include, in addition to the construction costs of the dam and appurtenant structures, the probable cost of lands, water rights (if existing rights must be purchased or subordinated), rights-of-way, and clearing the reservoir areas; costs of relocating public highways, railroads, buildings, and other properties; and engineering and administrative costs. Estimates will also be needed for annual costs for financing and for operation, maintenance, and replacement. The feasibility estimate may not be in full detail, but in overall amount it should represent a ceiling within which the project features can be built, barring significant advances in unit prices. The final estimate will be based on the subsequent detailed studies made in connection with the preparation of specifications and should be in sufficient detail to serve as a guide in securing bids and awarding a contract for construction.

27. *Quantity Estimates.*—After details of the de-

sign of the dam and auxiliary structures have been determined, the overall design can be completed and construction quantities computed. In preparing estimates for excavation and embankment work, allowances must be made for wasted and unsuitable materials, shrinkage from excavation to compacted fills, swell of rock excavation, and overbreak in tunnel and channel excavation. The contingency percentage usually added to an estimate is for the purpose of covering unforeseen difficulties, changes of plans, and detailed items of design that may be changed or possibly omitted because of limited funds. This contingency percentage does not cover overexcavation or excessive waste in construction.

In preparing estimates, correction factors or percentages should be applied to net computed quantities for certain classes of work. The major items likely to be in error are embankments, excavation, and concrete quantities. Soil for compacted earth embankments, if measured in borrow excavations will usually shrink from 10 to 30 percent when compacted—in general, about 15 percent. An allowance of 20 to 40 percent, generally 25 percent, for the swell of rock from excavation to rockfill and riprap should be made. Adequate allowance for overbreak should be made for excavation and for concrete lining in tunnels. Overbreak will vary with the tunnel size; with the nature of the material as to its composition, blockiness, and laminar makeup; and with the construction methods employed. Overbreak in tunnels can amount to as much as 40 percent of the minimum bore quantities. Additional concrete lining quantity percentages in such instances will be considerably higher, since the ratio will be based on the area of the concrete ring rather than on the entire bore area. An allowance must be made also for cement and aggregate wastes in the concrete quantities.

The estimator must be generous in computing quantities and yet avoid unreasonable and excessively high estimates. Laboratory data available are of considerable importance to the estimator in making the proper allowances for the estimated quantities. This is especially true of embankment materials and concrete mixes. An important feature of estimating quantities is a general understanding of the definitions for the various items with respect to specifications or pay dimensions. In preparing a schedule of quantities it is generally advisable to list all items on a standard form and according to standard specifications so that the items may be carefully checked against quantities and so that omissions may be avoided.

28. ***Unit Costs.***—Because of widely varying economic and labor conditions in different localities, it is not possible to furnish unit prices for use in estimating each class of work. Furthermore, unit costs fluctuate widely in response to economic conditions and other factors. The estimator should familiarize himself with local conditions, probable sources of materials and labor supply, cost of similar work in the locality, and probable changes in costs of materials and labor that may occur before actual construction due to economic adjustments. During periods of economic fluctuations, consideration should be given to construction cost indexes and trends, with an intelligent application to the particular project.

The costs of operations such as excavation, embankment, and mass concrete work have not risen in proportion to the increase in labor costs, because of improved methods and greater use of machinery and mechanical equipment. In fact, for some large-scale operations, unit costs have been reduced. On the other hand, the costs of construction operations involving a large percentage of labor (for example, reinforced concrete, formwork, and hand excavation) have greatly increased.

29. ***Finalizing the Project Plans.***—Plan formulation is a continuous process of coordination, analysis, and extension of all the specific studies directed toward achieving optimum size and scope of project and maximum benefits. It involves trial studies of various combinations of purposes and sizes and designs of structures so that in the final plans the sponsor may know, for example, that the area to be irrigated has been properly selected in consideration of the available water supply from various sources; that the reservoir is of proper size to achieve the best regulation of that water supply in terms of the investment for construction, operation, and maintenance; and that there are economic balances between spillway capacities and reservoir surcharge capacities, and between height of diversion dam and length of canal.

It should be emphasized that incorrect conclusions from the study are more often arrived at through forced interpretation of the data in favor of the project than through a lack of ability to evaluate properly those data. Under no con-

sideration should a factor be assumed as favorable until that assumption is supported by all available data.

In certain sections of the United States, water rights are commonly overlooked. Although no difficulties may have arisen in the past on a particular stream or in a particular area, water rights must be investigated for each proposed project.

The sanitary study will not be necessary for all projects. On the other hand, it is often incorrectly omitted in the study of projects where purity of water is a major consideration. Where such study is required, it is advisable to consider all potential sources of pollution in their most unfavorable light.

Where competent geologists are available, the geologic study will offer few difficulties. If such assistance is not available, the greatest caution should be exercised in interpreting geologic characteristics.

When thorough consideration has been given to each of the component studies for a particular project, they should each be briefed, listing: (1) The favorable, and (2) the unfavorable circumstances with regard to the project. Technical honesty should guide in the evaluation, and any unsoundness of the project should be recognized if soundness is not proved beyond a reasonable doubt.

30. ***Planning Reports.***—A reconnaissance report is generally prepared by the investigating engineer to make a record of the data available, to present a preliminary concept of the project plan with a rough economic and financial analysis, and to draw conclusions as to whether the project, based on data at hand, merits further study. If the recommendation is favorable, the report should outline the feasibility grade investigation to be made, the estimated costs and time required, and the requirements for personnel, equipment, etc.

The project feasibility report is generally prepared on completion of the feasibility investigations as a basis for advising the sponsor or owner and others who must approve or authorize the project of its merits. The report describes the project plans, features, costs, benefits, relationships to existing and future developments, problems, and financing. It should present definite recommendations, based upon probable accomplishments, regarding feasibility and acceptability under possible means of financing construction. The conclusions and recommendations should be adequately supported by the investigations, as documented in the report or its appendixes, in such form that, if necessary, the work may be readily reviewed by the proper authorities.

A feasibility report or a final report of a project involving the construction of a dam should include, as part of the report or as an appendix, a separate report on the design of the dam and its appurtenant structures. An outline for a report on the design of a dam is given in the following section.

31. ***Report on Design of Dam.***—A complete report should be prepared covering the investigation, engineering features, and cost of the proposed dam and reservoir. It should contain a general description of the design, including the various factors involved, a copy of the detailed estimate, and a drawing showing the general plan and sections. Included on the drawing should be a location map and curves showing hydraulic capacities.

To insure a complete description and record of all essential data, calculations, and conclusions entering into the design, a uniform procedure is desirable. The following outline of the items which the report should cover is included as a guide. Obviously, all of the information listed in this outline is not necessary for any particular small dam, but the greater part of it will be required for the larger and more complex structures.

__________ PROJECT

FEASIBILITY (OR SPECIFICATIONS) DESIGN AND ESTIMATE FOR __________ DAM

A. *Location and Purpose:*
1. Section, township, range, principal meridian, county, State, nearest city.
2. Location in respect to other features.
3. Accessibility.
4. Purpose:
    (a) Amount of storage—live, dead.
    (b) Type of storage—irrigation, flood, power, domestic, etc.
    (c) Water surface elevations.
    (d) Place where water will be used.
5. Alternate designs, if any.

B. *Summary of Design:*

1. Storage capacity ______ acre-feet.
2. Spillway capacity ______ second-feet at water surface elevation ______.
3. Outlet capacity ______ second-feet at water surface elevation ______.
4. Power outlet capacity ______ second-feet at water surface elevation ______.
5. Top of dam ______ elevation ______
6. Normal water surface ______ elevation ______
7. Maximum water surface ______ elevation ______ surcharge ______ acre-feet.
8. Minimum water surface ______ elevation ______
9. Freeboard above maximum high water. ______ feet.
10. Maximum height of dam above streambed. ______ feet.
11. Estimated cost of dam (or dam and reservoir). $______.
12. Estimated cost per acre-foot of storage. $______.
13. Total estimate, project ______ $______.
14. General plans and sections ______ Drawing No.______

C. *Design Data:*

1. Topography:
   - (a) Scale.
   - (b) Contour interval.
   - (c) Planetable sheet numbers.
   - (d) Surveyed by.
   - (e) Date of survey.
2. Geological report—author and title.
3. Logs of test pits and drill holes.
4. Hydraulic data, capacities and requirements and by whom established:
   - (a) Storage, irrigation, flood, power.
   - (b) Spillway.
   - (c) Outlet.
   - (d) Diversion.
   - (e) Area-storage capacity curves for various elevations of water surface.
5. Hydrologic data:
   - (a) Hydrographs.
   - (b) Maximum recorded flood.
   - (c) Inflow design flood.
   - (d) Mean annual runoff of drainage basin.
   - (e) Tailwater curve.
   - (f) Cross sections of streambed.
   - (g) Design values.
   - (h) Climatic conditions.
6. Borrow areas and aggregate deposits, location, and transportation facilities available:
   - (a) Laboratory tests.
7. Right-of-way information.
8. Photographs.

D. *Reservoir Data:*

1. Proposed capacities with corresponding water-surface elevations.
2. General dimensions.
3. Existing structures affected.
4. Nature of land flooded and clearing required.
5. Relocations: Railroad, highway, telephone lines, oil lines, powerlines.
6. Limitations to maximum reservoir flow line.
7. Geology:
   - (a) General formations.
   - (b) Factors relating to reservoir losses.
   - (c) Contributory springs.
   - (d) Deleterious mineral and salt deposits.
8. Right-of-way.

E. *Damsite Data:*

1. Geological features, formations:
   - (a) Nature of streambed and abutments.
2. Interpretation of test pits and drill holes.
3. Percolation tests, ground water.

F. *Dam Design:*

1. Number and types of estimates prepared.
2. Features governing design.
3. Drawing number.
4. Water-surface elevations, storage capacities, freeboard.
5. General dimensions:
   - (a) Top width.
   - (b) Description of section—slopes, height, zoning, etc.
   - (c) Crest length; roadway.
   - (d) Length of base at maximum section.
6. Percolation factor; sliding factor.
7. Cutoff trench and cutoff wall dimensions.
8. Grouting requirements.
9. Toe drains, drain holes.
10. Parapet and curbs.
11. Galleries.
12. Fishways, logways, etc.

G. *Outlet Works Design:*

1. Requirements:
   - (a) Discharges and corresponding water surface elevations.
   - (b) Diversion capacities and water surface elevations.
2. Factors affecting location.

3. Tunnel dimension—material encountered; liner plates.
4. Conduit dimensions.
5. Gate chamber:
   (a) Dimensions.
   (b) Location.
   (c) Accessibility.
6. Gates, valves, and pipes:
   (a) Dimensions.
   (b) Elevations.
7. Approaches, shafts, adits, plugs.
8. Location of controls.
9. Trashrack.
10. Stilling basin.

H. *Spillway Design:*
1. Requirements.
2. Factors governing design and location.
3. Type and description:
   (a) Controlled or uncontrolled.
   (b) Lining.
   (c) Dimensions.
   (d) Elevations.
4. Gates, gate structure:
   (a) Dimensions.
   (b) Operation.
5. Stilling basin:
   (a) General description.
   (b) Dimensions.
6. Approach and discharge channels.

I. *Construction Facilities:*
1. Estimated time to complete.
2. Power available.
3. Construction railroad, shipping points, hauls.
4. Construction camp.
5. Local conditions.

J. *Materials and Unit Prices:*
1. Location of borrow, hauls.
2. Aggregate deposits, hauls.
3. Cement, nearest mill, hauls.
4. Railroads, terminals.
5. Basis for unit prices.

C. BIBLIOGRAPHY

32. *Bibliography.*

[1] U.S. Department of Agriculture, Soil Conservation Service, "Rates of Sediment Production in Southwestern United States," January 1945.

[2] ———, "Reservoir Sedimentation in the Sacramento-San Joaquin Drainage Basins, California," July 1947.

[3] ———, "Rates of Sediment Production in Midwestern United States," December 1948.

[4] ———, "Sediment Design Criteria for the Missouri Basin Loess Hills," October 1950.

[5] ———, "Relation of Sedimentation to Accelerated Erosion in the Missouri River Basin," July 1951.

[6] ———, "Rates of Sediment Production in the Western Gulf States," March 1956.

[7] U.S. Department of Interior, Bureau of Reclamation, "Analysis of Flow-Duration, Sediment-Rating Curve Method of Computing Sediment Yield," April 1951.

[8] "Trap Efficiency of Reservoirs," Trans. Am. Geophys. Union, vol. 34, June 1953, pp. 407–418.

[9] U.S. Department of the Interior, Bureau of Reclamation, "Interim Report, Distribution of Sediment in Reservoirs," June 1954.

[10] Borland, W. M., and Miller, C. R., "Distribution of Sediment in Large Reservoirs," ASCE Proceedings, vol. 84, April 1958, Journal of the Hydraulics Division, No. HY2, pt. 1, paper No. 1587.

Flood Studies

D. L. MILLER AND R. A. CLARK[1]

A. GENERAL

33. ***Scope.***—This chapter considers methods of determining flood flows to be expected from the drainage area tributary to the reservoir site, for which provision must be made in the design of a dam. The flood used for design against failure is termed the "inflow design flood." In most instances, particularly for structures impounding considerable storage, the inflow design flood is the maximum probable flood, which is defined as the largest flood that can reasonably be expected to occur on a given stream at a selected point. Adoption of an inflow design flood less than the maximum probable flood is a policy decision to be made by the owner, agency, or organization responsible for construction of the project. Factors to be considered in arriving at such a decision are discussed in chapter VIII.

Determination of the maximum probable flood is based on rational consideration of the chances of simultaneous occurrence of the maximum of the several elements or conditions which contribute to the flood. A major consideration is the determination of the runoff that would result from an occurrence of a probable maximum storm based on meteorological factors. This hydrometeorological approach is necessary because streamflow records are of such relatively short duration in the United States that statistical analyses thereof do not provide reliable bases for estimates of maximum probable flood flows.

In addition to determination of inflow design floods, this chapter will indicate methods of determining the magnitude and frequency of floods as indicated by statistical analyses of streamflow records. These are primarily for use in connection with estimating diversion requirements during construction, establishing frequency of use of emergency spillways used in conjunction with outlets or small spillways, determining peak discharge estimates for diversion dams, or providing other information useful to the designers. Discussions in part A of this chapter are general; details of suggested procedures are given in part B.

34. ***Streamflow Data.***—The hydrologic data most directly useful in determining floodflows are actual streamflow records of considerable length at the location of the dam. Such records are rarely available. The engineer should obtain the streamflow records available for the general region in which the dam is to be situated. Locations of stream gaging stations and precipitation stations in the United States are shown on a series of maps entitled "River Basin Maps Showing Hydrologic Stations," edition 1949, prepared under the supervision of the U.S. Weather Bureau. Such data collecting stations are subject to change in location, discontinuation, or initiation of new stations. These maps cannot be kept current, and information thereon must be supplemented by additional investigations in order to be sure of the location and operation of stations in a given area. The engineer should consult the water supply papers and indexes of the U.S. Geological Survey[2] and, if possible, confer with the survey's district engineer. He should also make a search of the records of other Federal agencies which may have collected information in the region, and the records of State water conservation agencies or State geological surveys; and he should determine whether any

[1] Engineers, Flood Hydrology Section, Bureau of Reclamation.

[2] Published by the Government Printing Office and available in libraries designated as depositories of Government publications; most important libraries in the United States are so designated.

information may be available from other State departments, from county engineer offices, from municipalities in the vicinity, or from utility companies. Where streamflow records are not available, some agencies or inhabitants of the vicinity may have information about high-water marks caused by specific historic floods.

With respect to the character of the streamflow data available, flood flows at the damsite may be determined under one of the following conditions:

(1) *Streamflow record at or near the damsite.*—If such a record is available and covers a period of 20 years or more, the floodflows shown by the record may be analyzed to provide flood frequency values (sec. 54). Hydrographs of outstanding flood events can be analyzed to provide runoff factors for use in determining the maximum probable flood (secs. 46, 48, 50, and 51).

If such a record is available but covers only a few years, it may not include any flood of great magnitude within its limits and, if used alone, it would give false indication of flood potential. Analysis may, however, give some or all of the runoff factors needed to compute the maximum probable flood. Frequency values obtained from a short record should not be used without analysis of data from nearby watersheds of comparable runoff characteristics.

(2) *Streamflow record available on the stream itself, but at a considerable distance from the damsite.*—Such a record may be analyzed to provide unitgraph characteristics (secs. 47, 48, and 49), and frequency data which may be transferred to the damsite by appropriate area and basin-characteristic coefficients (secs. 49 and 54). This transfer can be made directly from one drainage area to another if the areas have comparable characteristics. Often damsites are located within the transition zone from mountains to plains and the stream gaging stations are located well out on the plains; in such instances, special care must be exercised when using the plains record for determination of floodflows at the damsite.

(3) *No adequate streamflow data available on the specific stream, but a satisfactory record for a drainage basin of similar characteristics in the same region.*—Such a record may be analyzed for unitgraph characteristics and frequency data, and these data transferred to the damsite by appropriate area and basin-characteristic coefficients.

(4) *Streamflow records in the region, but not satisfactorily useful for application and analysis under one of the above methods.*—These records may be assembled and analyzed as reference information on general runoff characteristics.

(5) *Use of high-water marks.*—High-water marks pointed out by inhabitants of the valley should be used with caution in estimating flood magnitudes. However, where there are a number of high-water marks in the vicinity of the project, and particularly if such marks are obtained from the records of public offices (such as State highway departments or county engineers), they may be used as the basis of a separate supplemental study. These records may be used to determine the water cross-sectional area and the water surface slope for the flood to which they refer, and from these data an estimate of that particular flood peak may be prepared using the slope-area method described in appendix B.

Whenever it appears that there will be one or more flood seasons between the selection of the damsite and construction of the dam, facilities for securing a streamflow record for the project should be set up as promptly as possible. This is of particular importance in order to obtain watershed data directly applicable to the computation of the inflow design flood for the dam, although a record usable for frequency computations cannot be secured. The facilities for obtaining such a record should be the best possible depending on the circumstances. A detailed discussion of these facilities, which may consist of either nonrecording or recording gages, is included in the following publications: "Equipment for Current-Meter Gaging Stations," U.S. Geological Survey Water Supply Paper 371; "Stream-Gaging Procedure," U.S. Geological Survey Water Supply Paper 888; and "Stream Flow," by Grover and Harrington, John Wiley & Sons, Inc., New York, 1943. The advice of Geological Survey engineers will be helpful in the site selection and installation, operation, and interpretation of records obtained.

The importance of utilizing records of runoff originating from the watershed above the damsite cannot be overemphasized. In the case of a damsite located on an ungaged stream, the establish-

ment of measuring facilities as discussed above may produce basic data which would justify "eleventh hour" revision of the plans, thus improving the design of the dam.

35. ***Precipitation Data.***—In each of the situations outlined in the preceding section, precipitation data are needed to evaluate factors for use in computing the maximum probable flood. The engineer should assemble the information with respect to precipitation during the greater storms in the region, and particularly for those storms for which runoff records are available. Such information can be obtained from publications of the U.S. Weather Bureau. At present (1958), daily precipitation data for each month for each State are contained in the publication "Climatological Data." Hourly data for each month for each State obtained by recording precipitation gages are contained in the publication "Hourly Precipitation Data."[3] Often precipitation data obtained by Weather Bureau precipitation stations have been supplemented by "bucket survey" data, i.e., information on rainfall amounts of unusual storms obtained from residents within the storm area by personnel of the Weather Bureau and other Government agencies.

Locations of precipitation stations as of 1949 are shown on the series of maps "River Basin Maps showing Hydrologic Stations," previously referred to. The need may arise for utilizing frequency values of rainfall (sec. 42). Weather Bureau publications presenting these data are included in the bibliography, section 55.

If plans are made to install streamflow measuring facilities as discussed in the preceding section, provision should also be made for obtaining precipitation records. An important item to consider is the selection of the location (or locations) of the precipitation gage, so that the catch will be a representative sample of average precipitation over the watershed. A comprehensive discussion of types of precipitation gages and observational procedures is contained in the U.S. Weather Bureau publication "Instructions for Climatological Observers," Circular B, 10th edition, 1952.

36. ***Use of Streamflow and Precipitation Records.***—From a flood hydrologist's point of view, an objective of analyzing streamflow and precipitation records is the development of procedures whereby the hydrograph (time versus distribution of runoff) that will result from a given amount of rainfall may be estimated. Another objective is the computation of a flood magnitude-frequency relationship based on experienced events. Certain procedures which have been developed for attaining these objectives have been selected for presentation in this text as applicable to problems encountered in the design of small dams. Engineers interested in a full discussion of the subject are referred to the texts included in the bibliography, section 55.

The primary use of recorded flows and precipitation records in the computation of an inflow design flood is the determination of a unitgraph and of retention loss characteristics of the watershed. Details regarding these analyses are presented in sections 46, 47, and 48. Where a streamflow record of sufficient length is available, a frequency curve may be computed by the procedure outlined in section 54

37. ***Watershed Data.***—All available information concerning watershed characteristics should be assembled. A map of the area above the damsite should be prepared showing the drainage system, contours if available, drainage boundaries, and locations of any precipitation stations and streamflow gaging stations. Available data on soil types, cover, and land usage provide valuable guides to judgment. Soil maps prepared by the Soil Conservation Service of the U.S. Department of Agriculture will prove helpful when the watershed lies within areas so mapped. The availability of this information is discussed in section 82.

The engineer preparing the flood study should make an inspection trip over the watershed to verify drainage area boundaries and soil and cover information, and to determine if any noncontributing areas are included within the drainage boundaries. The trip should also include visits to nearby watersheds if it is anticipated that records from nearby watersheds will be used in the study.

38. ***Factors to be Considered in Estimating Flood Flows.***—(a) *General.* Flood formulas primarily have been derived from and are directed toward peak discharge computations. Peak discharge values of major floods are more readily obtainable than volume values; hence, there are more data available for peak discharges. However, in most instances, the volume of runoff associated with

[3] Subscription to these publications may be made through the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C.

the peak discharge and its time distribution is of vital concern to the designers, who usually need a hydrograph of the inflow design flood. For this reason as well as for other reasons to be discussed later, flood formulas can be considered only as a guide to preliminary thinking and for making comparisons.

The detail with which hydrologic computations need be made in preparing a flood study depend on: (1) The character and applicability of the streamflow data available, and (2) the relationship of spillway cost to overall cost of the project. In regions of high rainfall potential, the cost of a spillway adequate to prevent failure sometimes makes construction of a dam exceedingly costly. For important projects, and particularly where the spillway cost is a major item of project cost and thus may have an important bearing on the feasibility of a project, the best possible use of streamflow data should be made. The hydrologic studies required in such instances are extremely complex, and consultation with an engineer or hydrologist experienced in this work is recommended. For small projects, or for those projects in which spillway capacity is obtainable at relatively low cost, a sufficient approximation of the inflow design flood discharge may be determined by procedures discussed in this chapter.

Statistical analyses of streamflow records do not provide reliable estimates of maximum probable flood discharges. Streamflow records provide the relationship between storm precipitation and runoff and information on runoff distribution. The determination of the maximum probable flood should be based on a study of storm potential, runoff potential, and runoff distribution as related to the physical characteristics of the watershed.

Each stream and each gaging station of a drainage area presents an individual problem because of the varying rainfall distribution and different runoff characteristics. It is not possible to express all factors affecting runoff in one simple formula for all watersheds. The simplicity of the Rational Method formula, $Q=CiA$, which is familiar to engineers encountering problems of runoff estimates, is deceptive in that a proper evaluation of the coefficient, C, and rainfall intensity, i, requires a detailed hydrologic study for each application.

For flood determinations, data concerning the following factors for each site need to be assembled and studied: Geographical location, storm potential, drainage area, soil and cover, and runoff distribution. A general discussion of each of these factors is given in the remainder of this section, and a procedure is presented in section 39 which provides a means of progressively evaluating each of these factors and obtaining a maximum probable flood hydrograph.

It will be apparent from the following discussions that determination of a maximum probable flood hydrograph can become quite complex, and that an expert in this field should be consulted in such determinations for important projects. Enumeration of the complexities which may be encountered are not attempted; texts on hydrology are listed in the bibliography, section 55.

(b) *Geographical Location.*—Flood potential varies greatly between geographical subdivisions due to differences in geology, topography, and moisture sources. There is a close relationship between geographical location and storm characteristics. Depending on location, floodflows originate from rainfall, from snowmelt, or from a combination of varying amounts of rainfall and snowmelt. Certain topographic features exert significant influence on precipitation amounts, and certain geological formations are conducive to high flood discharges while others tend to reduce flood potential. An engineer preparing floodflow estimates should be familiar with the effect of these factors for the area in which he is interested.

(c) *Storm Potential.*—The first practical application of meteorology to studies of probable maximum precipitation, dates from cooperative efforts by the U.S. Corps of Engineers and the U.S. Weather Bureau in 1937. Shortly afterward, the Hydrometeorological Section of the Weather Bureau was established to perform such duties on a continuing basis. A Cooperative Studies Section of the Weather Bureau made similar studies for the Bureau of Reclamation from 1946 to 1954. The several reports of these sections (listed in the bibliography, sec. 55) provide storm potential data for many parts of the United States.

As the field of hydrometeorology is relatively new, a certain amount of confusion as to exact meaning of its terminology is to be expected, particularly when terms such as "maximum probable," "maximum possible," "probable maximum," etc., may or may not be intended to represent a difference in storm magnitude. In this text, the terminology used in regard to storm

potential is in accordance with Weather Bureau practice. The terms "probable maximum precipitation" and "probable maximum storm" represent two different things. Probable maximum precipitation for a particular area represents an envelopment of depth-duration-area rainfall relations for *all* storm types characteristic to that area adjusted meteorologically to maximum conditions, whereas probable maximum storm values consider such rainfall relations for only *similar* storm types. Probable maximum storm values are usually smaller than probable maximum precipitation values because different storm types are not combined. This subject is further discussed in section 45.

Generalized charts for estimating probable maximum precipitation east of the 105° meridian are published in Hydrometeorological Report No. 33 of the Hydrometeorological Section, U.S. Weather Bureau, Department of Commerce. Bureau of Reclamation hydrometeorologists have prepared some 200 probable maximum storm estimates for watersheds in the mountainous regions of western United States in addition to numerous such estimates prepared by the Cooperative Studies and Hydrometeorological Sections of the Weather Bureau. On the basis of these data, generalized maps of storm potential (sec. 45) have been prepared for this text to make possible the derivation of inflow design floods for areas throughout the United States by the hydrometeorological approach. It is recognized that continuing development of storm study techniques will lead to revision of storm potential estimates. A general study of storm potential in western United States is now (1958) in progress as a cooperative endeavor of the U.S. Weather Bureau and the U.S. Soil Conservation Service.

(d) *Drainage Area.*—An accurate map of the drainage area above a point of interest is essential to any flood study. This subject has been discussed in section 37

(e) *Soil and Cover.*—The type of soil and vegetative cover of a watershed has a marked influence upon its runoff potential. It is difficult to express soil and cover types in numerical values which represent the difference between precipitation amounts and runoff amounts. If both precipitation and runoff data are available for several events, practical values can be obtained by direct analysis (sec. 46); if such data are not available, recourse must be made to comparisons with analyses for other watersheds. A method developed by the U.S. Soil Conservation Service for estimating runoff on the basis of soil type and cover is discussed in section 46 and is presented in detail in appendix A.

(f) *Runoff Distribution.*—The unitgraph has proved to be a very effective tool for hydrologic work. An engineer preparing flood estimates should be familiar with its basic principles. A brief outline of unitgraph derivation and application, including a dimensionless graph approach for derivation of a synthetic unitgraph for ungaged areas, is given in sections 47, 48, and 49. A concept of representing a unitgraph as a triangle and the relationships that can be derived therefrom is presented in section 50. Interested engineers are referred to the texts listed in the bibliography, section 55, for more detailed discussions of the unitgraph.

39. A Method of Computing Maximum Probable Flood Discharge.—The following discussion pertains in general to data contained in the "Hydrology Guide for Use in Watershed Planning," [4] published by the Soil Conservation Service. The introduction states: "The Hydrology Guide is intended for the use of Soil Conservation Service technicians and engineers. It presents material needed for fulfilling national service responsibilities in the field of soil and water conservation. Other technicians and engineers may find portions of the Guide useful in their own work, but they should recognize that the Guide attempts first to fill Service needs and requirements."

The Soil Conservation Service procedures are based on conclusions regarding applicable average values obtained from analyses of many natural flood events and are used to evaluate runoff for several purposes including inflow design flood computations. The sequence for computing an inflow design flood as outlined by the SCS Guide has been adopted in this text, along with many of their conclusions regarding average applicable values for unitgraph determinations and retention rate estimates. However, the charts for obtaining design storm values (sec. 45) were prepared especially for this text to be more directly applicable to inflow design flood determinations, and are not the

[4] "Hydrology Guide for Use in Watershed Planning," *National Engineering Handbook*, Sec. 4, Hydrology, Supplement A, U.S. Department of Agriculture, Soil Conservation Service.

same as presented in the SCS Guide. Also, a minor modification of the SCS method of estimating direct runoff from design storm precipitation has been made as discussed in section 53.

The method presented herein is applicable to watersheds *where flow originates principally as overland (direct) runoff from precipitation in the form of rain.* Those watersheds having runoff-delaying media such as lakes, swamps, heavy forest litter, and porous top soil require special study. Watersheds from which maximum runoff will likely include snowmelt also require special study. It is beyond the scope of this text to discuss these special studies in detail.

This procedure for computing a maximum probable flood is based on the hydrometeorological approach and requires estimates of storm potential and the amount and distribution of runoff. As in all generalized procedures, certain criteria have been adopted arbitrarily as applicable to the greatest number of cases. A step-by-step explanation of the procedure and computations for an example are given in section 53. A brief description of the procedure follows:

(1) A 6-hour point rainfall value is obtained for the geographical location from an appropriate chart.

(2) By means of graphs, this point rainfall value is adjusted to represent 6-hour average precipitation over the subject drainage area and also adjusted to give the cumulative rainfall for longer durations up to 48 hours.

(3) Charts and tables are used to determine the time-distribution of the rainfall.

(4) Soil and cover data are translated by means of tables into an appropriate runoff curve number from which the runoff volume is determined for increments of time D in equation (1).

(5) Based on the concept of representing hydrographs as triangles, a triangular hydrograph for 1 inch of runoff (unitgraph) from the subject watershed is computed from the following relationships:

$$T_p=\frac{D}{2}+0.6T_c \qquad (1)$$

$$T_b=2.67T_p \qquad (2)$$

$$q_p=\frac{484AQ}{T_p} \qquad (3)$$

where:

T_p=time to peak, hours,
D=rainfall excess period, hours,
T_c=time of concentration in hours, defined as travel time of water from hydraulically most distant point in the watershed to the point of interest,
T_b=time length of base of hydrograph, hours,
q_p=peak discharge in second-feet,
Q=volume of runoff, in inches, and
A=area of watershed in square miles.

Development of the above equations is presented on figure 12.

(6) The triangular hydrograph for each interval, D, is computed by direct ratio of the runoff for that interval to 1 inch of runoff and plotted.

(7) The total runoff hydrograph is obtained by graphical addition of the incremental hydrographs.

This procedure might be termed a "generalized" unitgraph approach. An engineer using the above equations should take cognizance that the relationships of time-to-peak and base-time expressed by the equations represent average values for these relationships as adopted from numerous analyses of natural flood events made by Soil Conservation Service hydrologists.

If reliable streamflow hydrographs are available, analyses can be made to determine relationships expressed by the above equations that are applicable to the subject watershed, and these should be used. For ungaged watersheds, the above equations should give usable values for rain floods if the watershed has no unusual runoff characteristics. Representing the hydrograph as triangular instead of a natural curvilinear shape reduces computations. The error in shape thus introduced results in a slightly more severe hydrograph in that a triangle distributes a given amount of runoff in a shorter time interval than does a curvilinear hydrograph. (See fig. 14(A).)

40. *Inflow Design Flood.*—As stated in section 33, the inflow design flood is usually equivalent to the maximum probable flood, but in certain instances, as discussed in chapter VIII, a flood of smaller magnitude than the maximum probable may be selected for use as an inflow design flood. Floods smaller than the maximum probable can

be computed by using design precipitation values less than the probable maximum without changing runoff factors, or by using lesser design precipitation values in conjunction with varying degrees of runoff potential as determined from antecedent moisture supply. Procedures for computing floods less than the maximum probable are presented in section 53.

Before a decision is made as to the exact magnitude of the inflow flood to be adopted for design, a comparison should be made between computed synthetic values and records of floods that have occurred in the general area of interest. A method of making such comparisons is discussed in the following section.

41. *Envelope Curves.*—Peak discharge envelope curves and flood volume envelope curves can be prepared by drawing curves enveloping plotted points representing maximum recorded values for various drainage areas. The values plotted should represent similar type floods (rain floods or snowmelt floods) that have occurred within the broad geographical subdivision within which the subject watershed lies, and should not be limited to events of a single small river system. Preparation of envelope curves for a general area provides an engineer with valuable information on past flood history and an indication of the flood of record comparable to the subject area. However, they should not be relied upon as a means of estimating maximum probable flood values. Design flood values purporting to be the maximum probable should be higher than those obtained from envelope curves. Only in specific instances where a watershed has definitely lower flood potential than neighboring watersheds due to soil type, surface storage, etc., would it be good judgment to adopt an inflow design flood of smaller magnitude than that of a flood which has occurred nearby.

A simple method of preparation of envelope curves is to tabulate maximum peak discharges (or volumes of a selected duration) and respective drainage areas prior to plotting points. In most instances, the drainage area above a stream gaging station or the point of a large flood discharge measurement is given in the United States Geological Survey water supply paper listing the flood. When it is known that only a portion of the drainage area above a point of measurement contributes to a flood, the size of that contributing portion should be used in the envelope curve analysis. Discharges or volumes are plotted versus respective drainage areas using log-log paper. Data thus plotted usually indicate a curved line envelopment on log-log paper which may be approximated by a straight line for small ranges in areas. High discharges from local thunderstorms may suggest consideration of two curves—one for smaller areas subject to such occurrences and another for larger areas where maximum discharges originate from general storms.

42. *Estimates of Frequency of Occurrence of Floods.*—Estimates of the magnitude of floods which have frequencies of 1 in 5, 1 in 10, or 1 in 25 years are helpful in estimating requirements for stream diversion during construction. These floods are normally termed the "5-, 10-, or 25-year flood." The magnitude of more rare events such as the 50- or 100-year flood may be required for reasons such as to establish sill location of emergency spillways, to design diversion dams, etc. The usual term of expression, "*x*-year flood," should not lead to the wrong conclusion that the event indicated can happen only once in *x* years, and having occurred, will not happen again for another period of *x* years. It does mean that over a long span of years we can expect as many *x*-year floods (or larger) as there are *x*-year-long periods within that span. Floods occur randomly and may be bunched or spread out unevenly with respect to time. No predictions are possible for determining their distribution; the maximum probable flood *can* occur the first year after the project is built, though of course, the odds are heavily against it.

The frequency of a flood should be considered as the chances of occurrence of a flood of that size (or one larger) in any one year. Stated another way, the chances of the flood in any one year being equaled or exceeded by floods of the magnitudes indicated as the 5-, 10-, 25-, or 100-year floods have ratios of 20:100, 10:100, 4:100, and 1:100, respectively.

Many methods of flood frequency determinations based on streamflow data have been published. An excellent summary of these methods along with comments on factors affecting their accuracy and limitations is contained in a

paper entitled "Review of Flood Frequency Methods."[5]

If streamflow data for a period of 20 years or more are available for the subject watershed or comparable watersheds, frequency curve computations yield acceptable results for estimates up to the 25-year flood and may be extrapolated to indicate the 100-year flood with fair assurance of obtaining acceptable values. Hazen's Method (see bibliography, sec. 55) is suggested for utilizing streamflow data to obtain frequency values. A procedural outline for such computations is presented in section 54. The hydrograph of a particular frequency flood is usually sketched to conventional shape using the peak discharge value and corresponding volume value obtained from computed frequency curves. In some instances, a peak discharge and associated volume of a recorded flood will correspond closely with a particular frequency value, in which case the recorded flood hydrograph is used.

For watersheds where runoff originates from rainfall and for which streamflow data are not available (usually small watersheds), an indication of flood frequencies can be obtained by estimating probable runoff from precipitation data of the desired frequency. Probable rainfall intensities for short durations can be obtained from U.S. Weather Bureau publications (see bibliography, sec. 55), or in some instances by direct frequency analyses of records at nearby precipitation stations. These data provide means of obtaining probable "x-year" precipitation values for various time durations. A storm duration time is assumed equal to that of the time of concentration, T_c, estimated for the watershed. Triangular hydrographs representing the desired frequency flood are computed by the procedure presented in section 53. The uncertainties inherent in estimating the amount of runoff which will result from a given amount of rainfall make the results of this procedure less reliable than that of using streamflow data where such data are available.

43. *Special Cases.*—The discussions in this chapter have dealt primarily with floodflows resulting from runoff from rain falling on an unfrozen watershed. There are areas within the United States where large floods have originated from rapid snowmelt on a frozen watershed or from a combination of rain falling on a snow cover over a frozen watershed. The eastern and western coastal regions of the United States are subject to floods caused by rain falling on a snow cover over watersheds having usually unfrozen soil. These types of events require special study to ascertain the amount of water that becomes available for runoff. Once that is determined, distribution of the runoff by a unitgraph usually is feasible. Although such special cases may be rare within an area of interest, they may represent the maximum probable flood-producing conditions.

Instances in which snowmelt runoff provides the major portion of a maximum probable flood usually involve major streams and large drainage areas. Design of structures for such is beyond the scope of this text. However, problems may arise concerning small mountainous watersheds where snowmelt runoff is a dominant characteristic. In these areas, the flow in the stream due to snowmelt runoff may be large and should be added to the computed rain flood. Also, when estimating runoff from rainfall in such areas, it should be recognizes that part of the watershed will be covered by melting snow which is satisfying retention losses under the snow-covered area. The runoff from rain falling on the snow-covered area may be equal to the rainfall. Therefore, the overall retention loss for rainfall on a partially snow-covered watershed during the melting season will be less than that for the same watershed when bare of snow.

It should be kept in mind that, although a usual sequence of events may produce floods, it is generally the unusual event or series of events that produce the great floods. The occurrence of two hurricane storms a few days apart following the same path over a large area in the northeastern States in August of 1955 is a prime example. Hasty conclusions as to flood potential should not be made on the basis of a long period of streamflow record. For example, although the recorded maximum peak discharge on the Trinity River at Lewiston, Calif., was 40,300 second-feet for the years 1911 through 1954, a discharge of 71,600 second-feet occurred on December 22, 1955.

[5] "Review of Flood Frequency Methods," Final Report of the Subcommittee of the Joint Division Committee on Floods, *Trans. ASCE*, vol. 118, 1953, pp. 1220–1231.

B. PROCEDURES

44. *Introduction.* —The selection of an appropriate inflow design flood is an essential part of the engineering studies for a project. The words "selection" and "appropriate" are used advisedly because a considerable amount of engineering judgment must be exercised in any hydrologic study of flood potential. It might be presumed that the problems of determining an inflow design flood would decrease in direct ratio to the size of the drainage areas involved and that such problems for drainage areas above small dams could be resolved quite easily. Such is not the case. In many instances, the hydrologic problems for small drainage areas are less easily resolved than those for large areas because relevant data for small natural watersheds are extremely meager.

It is believed that those using the material in this text most often will be concerned with projects for which little direct hydrologic data are available. Therefore, material and procedures presented herein have been selected with a view to assisting in solution of flood estimating problems for such projects. However, all available recorded streamflow and precipitation data should be utilized to the fullest extent possible, and outlines for methods of analyzing these data are included in this text. Discussions of procedural developments have been omitted or condensed, but the engineer so interested may obtain them from references cited in the bibliography, section 55. Discussions of the following subjects are presented:

45. *Estimating Storm Potential.* —(a) *General.*—An estimate of storm potential is an integral part of the hydrometeorological approach to computation of inflow design floods. The term "storm potential" is all inclusive, embracing factors such as rainfall intensity, duration and areal extent. Meteorologists are able to establish maximum values for these factors which, judged by present knowledge, appear to be the limit of nature's capabilities. These maximum values differ throughout the United States (and the world). Knowledge of such limits, and the resulting probable maximum precipitation, provides the hydrologist with a good starting point for his estimate of a maximum probable flood as well as for floods less than the maximum probable. The precipitation values adopted for computing the selected inflow design flood are usually referred to as design storm values.

(b) *Definitions.*—For the purposes of this text, the following terminology is used:

(1) *Probable maximum precipitation.*—Probable maximum precipitation values represent an envelopment of maximized intensity-duration values obtained from all types of storms. It is recognized that probable maximum precipitation values for all durations and for all areas will not occur from any one type of storm. For example, a maximized thunderstorm is very likely to provide probable maximum precipitation over an area of 50 square miles for a duration of 6 hours or less, but the controlling values for longer durations or for larger areas will almost invariably be obtained from general storms.

(2) *Probable maximum storm.*—The probable maximum storm values represent an envelopment of maximized intensity-duration values obtained from one type of storm only. For this storm, consideration is given to season, storm type, and variation of precipitation with respect to both time and location.

(c) *Probable Maximum Storm Considerations.*—Estimates of probable maximum storms are based on analyses which consist of three steps: (1) Determining the areal and time distribution of the larger storms of record in the general area; (2) maximizing these observed storms by increasing their values to their physical upper limit as determined from a consideration of their observed moisture content in relation to the probable maximum moisture content that could be associated with a similar storm condition; and (3) considering transposition of these storms. The results of the first step will indicate which storms are best suited for further analysis and can also be used in the hydrograph analyses to estimate average retention loss rates and hydrograph lag times.

In the second step, the relation between maximum moisture potential and the moisture charge

of the inflowing airmass is considered. Other factors that are effective in determining the efficiency of a storm in converting atmospheric moisture into precipitation have not been defined sufficiently at the present time to enable their use for making estimates of storm efficiency.

Storm transposition considered in the third step is based on the assumption that the location of a particular storm depended upon meteorological factors that could just as easily occur over other locations within given regions. Transposition is limited to regions subject to similar types of storms and not separated by major orographic features. Because the period of record for any particular drainage basin is generally quite short, the transposition of other storms within the same homogeneous meteorological and orographic area has the advantage of combining regional experience of a large number of storms.

(d) *Generalized Precipitation Charts.*—An engineer encountering a design flood estimating problem needs information regarding storm potential. Since such information pertains to magnitudes of storms which could occur from a more severe combination of meteorological events than has yet been observed, the engineer cannot make his estimate directly from recorded storm data. It is impossible to show all the refinements and variations that can influence the magnitude of design storm values for all individual locations within the United States on a generalized chart. However, broad areas do have like storm potential. Generalized charts have been prepared for this text to provide one means of rapidly determining design storm values for any specific area. The design storm values obtained from the generalized charts represent a reasonable upper limit and in most cases will exceed the values obtained for a specific watershed by a detailed hydrometeorological study. If such a study is desired because of the importance or size of the project, the services of a consulting hydrometeorologist should be secured.

Two generalized charts, one for the United States east of the 105° meridian and one for the United States west of the 105° meridian are given. Figure 1 shows probable maximum 6-hour precipitation values for 10-square-mile areas of the United States east of the 105° meridian. This chart is based on one presented in Hydrometeorological Report No. 33, prepared by the Hydrometeorological Section of the U.S. Weather Bureau in collaboration with the U.S. Corps of Engineers (see bibliography, sec. 55). These 6-hour values for 10-square-mile areas can be modified for durations in excess of 6 hours and for larger areas up to 1,000 square miles by use of figure 2 (two sheets). Because of the unlikelihood of a perfect strike of a storm center on any particular small basin, no variation is assumed between point and 10-square-mile precipitation. For durations shorter than 6 hours, the time distribution of precipitation can be obtained from curve C, figure 4. The values obtained from use of these charts represent probable maximum precipitation.

Figure 3 shows probable maximum 6-hour point storm values for areas of the United States west of the 105° meridian. This chart is based on the results of approximately 200 design storm analyses prepared by the Bureau of Reclamation for specific drainage basins west of the 105° meridian as well as consideration of numerous design storm analyses made by the Cooperative Studies and Hydrometeorological Sections of the Weather Bureau. The variable topography of this part of the United States greatly influences the storm potential and permits only limited transposition of storms. These point storm values can be applied to areas up to 1,000 square miles by use of the curve presented on figure 5. The 6-hour storm values can be extended for longer duration periods by multiplying the 6-hour value by the appropriate factors shown in table 1.

TABLE 1.—*Constants for extending 6-hour design storms in areas west of 105° meridian to longer duration periods*

Duration (hours)	*Constant**
8	1.16
10	1.31
12	1.43
14	1.50
16	1.56
18	1.62
20	1.68
22	1.74
24	1.80
30	1.95
36	2.10
42	2.25
48	2.38

*Multiply 6-hour point rainfall from fig. 3 by indicated constant.

For durations shorter than 6 hours, the time distribution of storm values can be obtained from the curves presented on figure 4. Values obtained

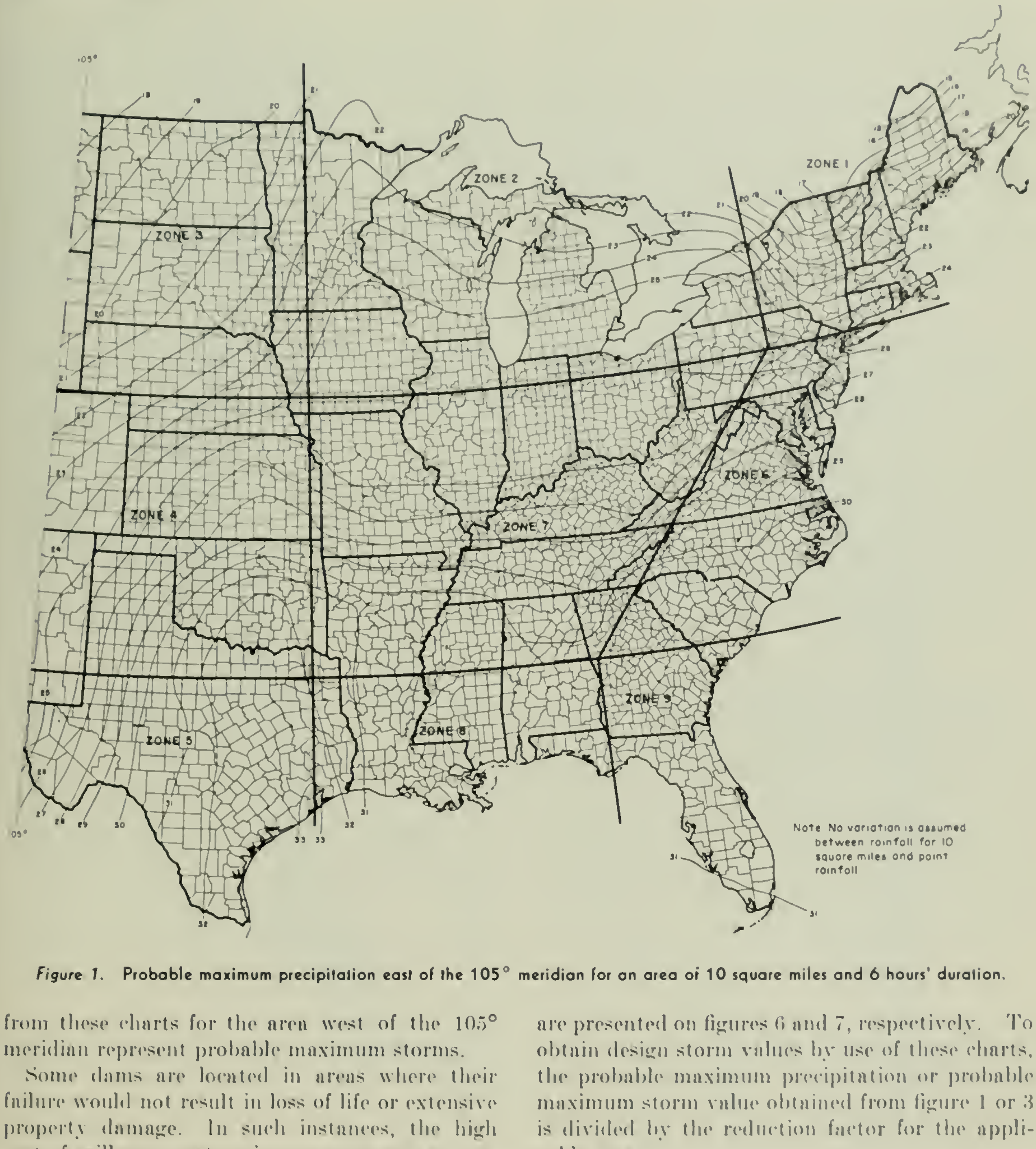


Figure 1. **Probable maximum precipitation east of the 105° meridian for an area of 10 square miles and 6 hours' duration.**

from these charts for the area west of the 105° meridian represent probable maximum storms.

Some dams are located in areas where their failure would not result in loss of life or extensive property damage. In such instances, the high cost of spillway construction may not warrant use of probable maximum precipitation in determination of the inflow design flood. Charts have been prepared indicating appropriate reduction factors to be applied to the probable maximum 6-hour values for determination of a design storm for use in a low hazard area. Appropriate reduction factors for areas east and west of the 105° meridian are presented on figures 6 and 7, respectively. To obtain design storm values by use of these charts, the probable maximum precipitation or probable maximum storm value obtained from figure 1 or 3 is divided by the reduction factor for the applicable area.

Design storm values obtained from figures 1 and 3 show considerable difference at their common boundary along the 105° meridian. This is due to the techniques used in determining the values shown on the charts. The values shown on figure 1 for the eastern United States are probable maximum precipitation based on envelopment of many

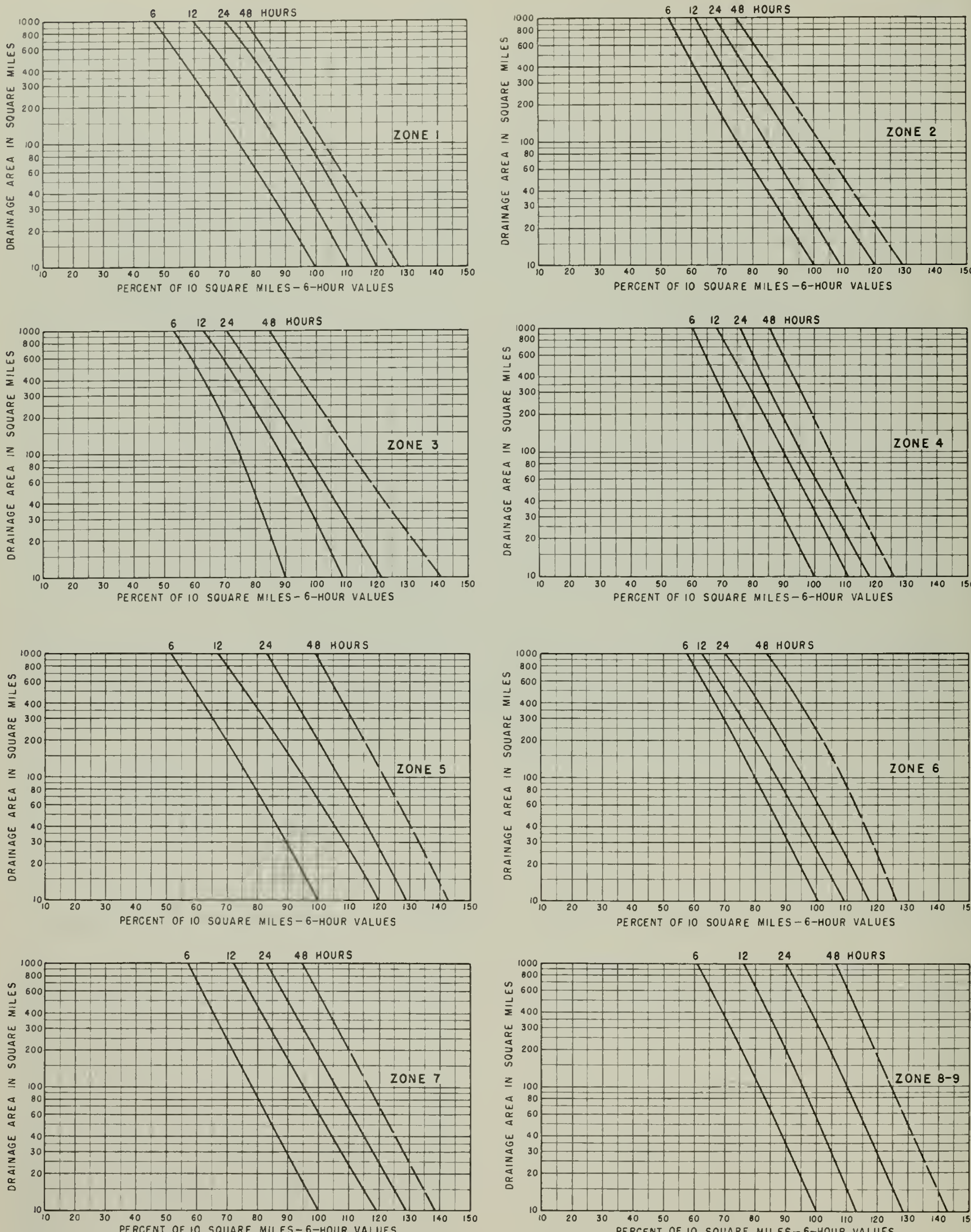


Figure 2. **Depth-area-duration relationships. Percentage to be applied to 10 square miles, 6-hour probable maximum precipitation values.**

Figure 3. Probable maximum 6-hour point storm values west of the 105° meridian.

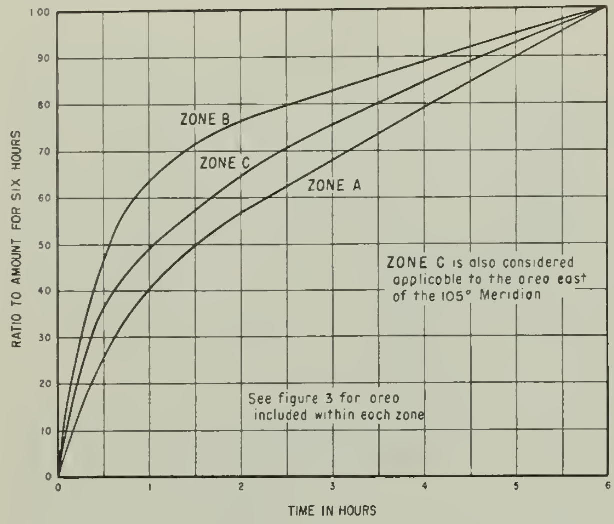


Figure 4. **Distribution of 6-hour rainfall for area west of 105° meridian.**

types of storms; whereas the values shown on figure 3 for the western United States are based on studies of probable maximum storms. Data have not been compiled for presentation of probable maximum storm values for areas of the United States east of the 105° meridian.

46. Estimating Runoff From Rainfall.—(a) *General.*—The hydrometeorological approach to analyzing flood events and predicting the magnitude of floods requires a firm estimate of the difference between precipitation and resulting runoff. From a flood determination point of view, this difference is considered loss, i.e., loss from the precipitation falling over a given watershed. A simple solution to derive this loss value appears to lie in finding the rate at which water will infiltrate into the soil. If this infiltration rate is known, along with the amount of precipitation, a simple subtraction should give the amount of runoff. However, there are other precipitation losses in addition to infiltration such as interception by vegetative cover, surface storage, and evaporation, that may have material effect on runoff amounts.

Various types of apparatus have been devised to test the infiltration rates of soils, and studies have been made of interception and evaporation losses. Although maps to an extremely large scale could define most of the surface storage area, it is apparent that an accurate volumetric evaluation of all the loss factors can be made only for a highly instrumented, small plot of ground and that such an evaluation is not practical for a natural watershed composed of many square miles of varying type soils, vegetative cover, and terrain features. For this reason, hydrologic literature contains many arguments against the "in-

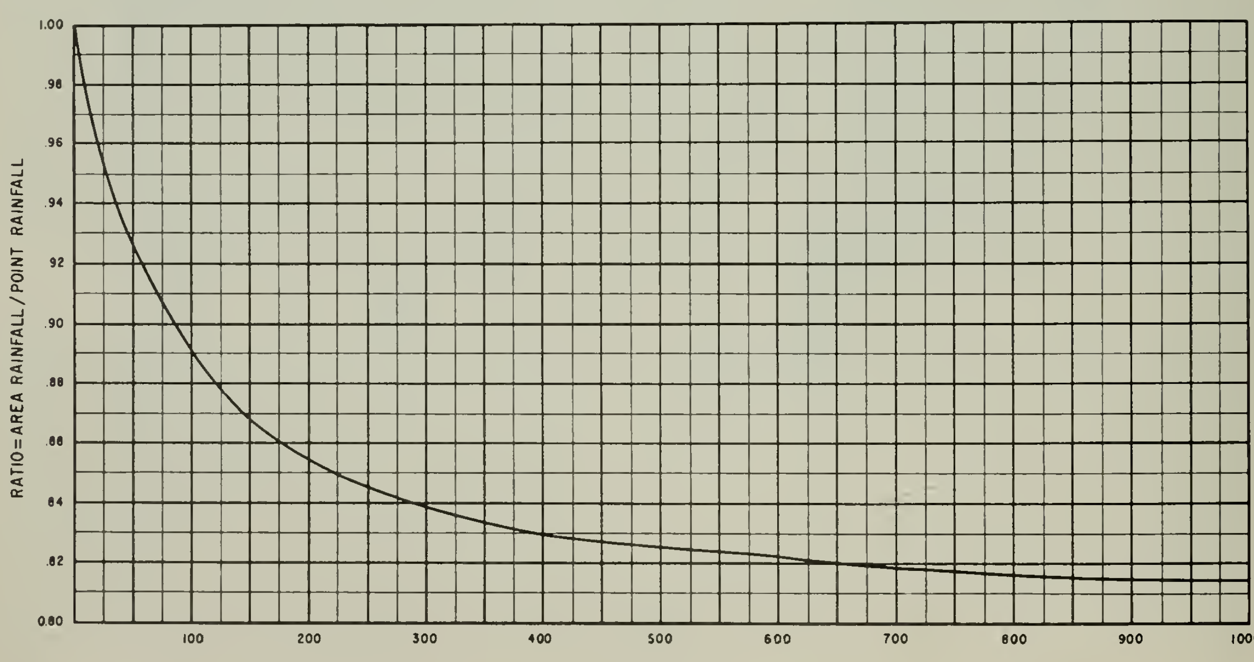


Figure 5. **Conversion ratio from 6-hour point rainfall to 6-hour area rainfall for area west of 105° meridian.**

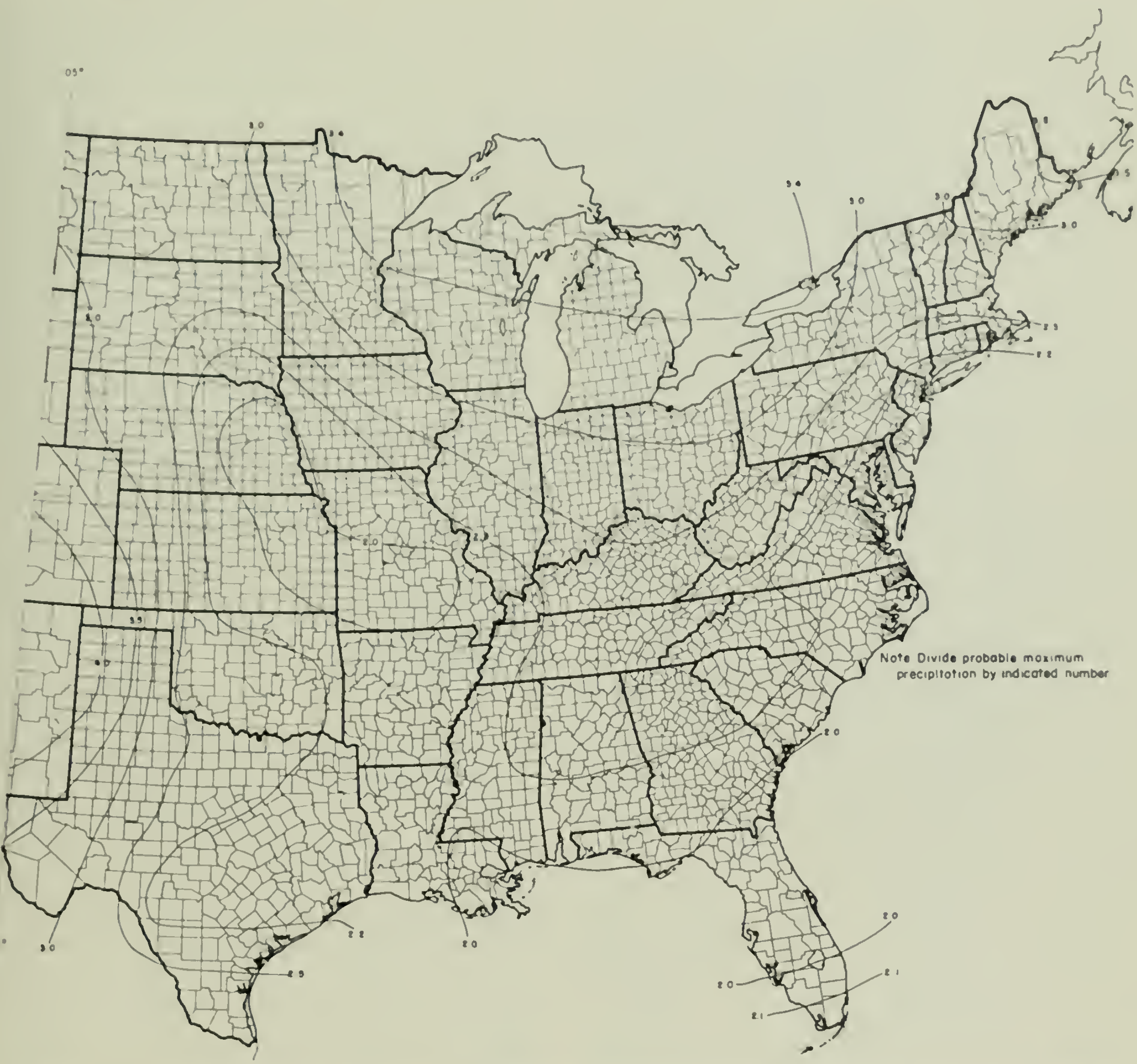


Figure 6. **Ratio for determining rainfall applicable for computing inflow design flood less than maximum probable for area east of 105° meridian.**

filtration rate approach" to determination of runoff amounts. However, the infiltration rate approach is applied on an empirical basis to obtain a practical solution to the problem of determining amounts of runoff, recognizing that the values used are of the nature of "index" values rather than "true" values. Natural events are studied and the difference between rainfall and runoff determined. Since this difference includes all the losses described above, it is usually called a retention loss or a retention rate. Such retention rates derived from available records may be adjusted to ungaged watersheds by analogy of soil type and cover.

An engineer making studies to compare rainfall with runoff must become familiar with the units of measurement used and the factors for conversion to common units. These conversion factors are given in appendix B. In the United States, precipitation is measured in inches and runoff is measured in cubic feet per second (also referred to as second-feet or sec.-ft.). It is necessary to know the watershed area contributing the runoff at a given measuring point in order to express the

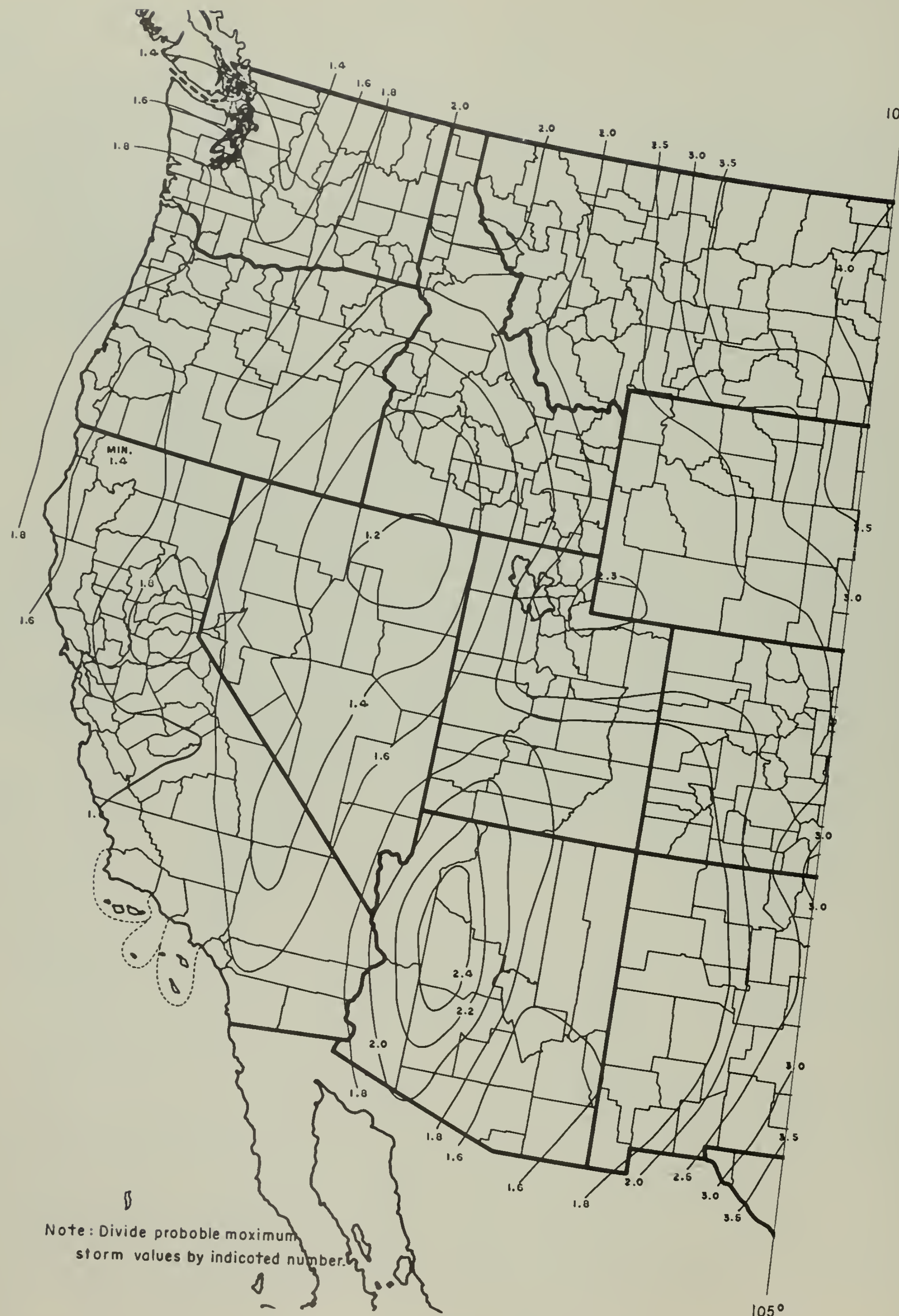


Figure 7. Ratio for determining rainfall applicable for computing inflow design flood less than maximum probable for area west of 105° meridian.

runoff volume in inches of depth over the watershed for comparison with precipitation amounts. When making such comparisons, the amount of runoff, expressed as inches, is termed rainfall excess, and the difference between the rainfall excess and the total precipitation is considered retention loss.

The discussion which follows describes a method of making a rainfall-runoff analysis. The objectives of such analyses are: (1) The determination of a retention rate, and (2) the determination of the duration time interval of rainfall excess. A comparison of retention rates derived from several analyses leads to adoption of a rate for design flood computations. The determination of the duration of excess rainfall is necessary for the hydrograph analyses computations involving determinations of unitgraphs and lag times, which are discussed in sections 48, 49, 50, and 52. In all such analyses, the runoff volume which is compared with precipitation amounts is that which relates directly to the rainfall under study. Therefore, the base flow of the streamflow hydrograph must be subtracted out before comparisons are made (see sec. 48).

(b) *Analysis of Observed Rainfall Data.*

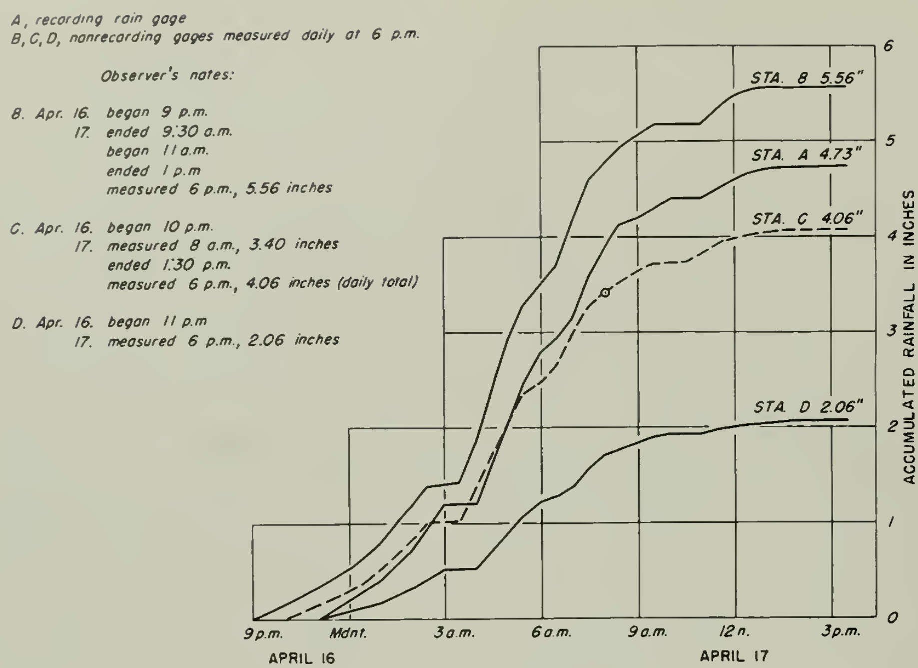
(1) *Mass curves of rainfall.*—Mass curves of cumulative rainfall during the storm period should be plotted for all precipitation stations in and near the basin as shown on figure 8(A). To show clearly the relation of rainfall to runoff, it is sometimes desirable to plot the mass curves to the same time scale as the discharge hydrograph of storm runoff. Usually, however, the curves should be given a more expanded time scale than it is desirable to use for the hydrograph analysis. When only one recording station is located nearby, and in the absence of better information, the mass curve of precipitation at a nonrecording station is usually considered to be proportional in shape to that of the recording station, except as otherwise defined by the observer's readings and notes (fig. 8(A)). The speed and direction of travel of the rainburst should be taken into account. Many rainfall observers enter the times of beginning and ending on the same line as the current daily reading. The notes may therefore refer to the previous day, especially when the gage is regularly read in the morning.

(2) *Isohyetal maps.* The total amounts of rainfall occurring during the portion of the storm that produced the flood hydrograph under study should be determined from the mass curves for each station in and near the drainage area. For a flood hydrograph consisting of a single event, this will be the total depth of precipitation occurring during the storm period. For a compound hydrograph, in which individual portions of the hydrograph are studied separately, temporary cessations of rainfall will usually be indicated in the mass curves, and from inspection it usually will be apparent which of the increments of rainfall caused the runoff event under study. The appropriate depths of rainfall are then used to draw an isohyetal map, using standard procedures. A typical isohyetal map is shown in figure 8(B).

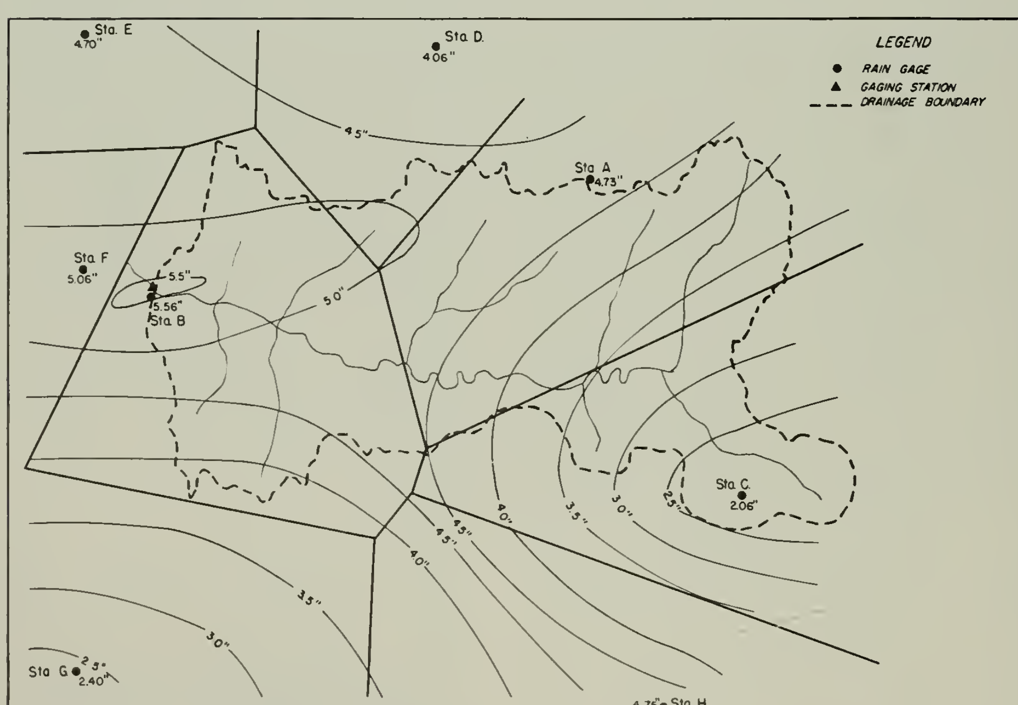
Extreme caution should be used in drawing the isohyetal pattern in mountainous areas where the orographic effect is an important factor in the areal distribution of rainfall. For example, if there is a precipitation station in a valley on one side of a mountain range and another station in a valley on the opposite side of the range with no intervening station, it cannot be assumed that the rainfall during a storm would vary linearly between the two stations. It is likely that the rainfall would increase with increases in elevation on the windward side of the divide, whereas on the leeward side, precipitation would decrease rapidly with distance from the divide. This type of distribution can usually be verified in mountainous areas where there are sufficient precipitation stations to define the isohyetal pattern accurately.

After the preliminary hydrographs and the isohyetal maps have been drawn, the atypical flood events for unit hydrographs determination may readily be eliminated. *Those floods having a combination of large volume, uniform intensities, isolated periods of rainfall, and uniform areal distribution of rainfall, should be chosen for further study.*

(3) *Average rainfall by Thiessen polygons.*—The average rainfall on a drainage area can be determined from precipitation station records by the Thiessen polygon method. A sample computation of average hourly rainfall from the mass curves in figure 8(A), using Thiessen polygons indicated in figure 8(B), is given in table 2.



(A) MASS CURVES OF RAINFALL



(B) ISOHYETS AND THIESSEN POLYGONS

Figure 8. Analysis of observed rainfall data.

TABLE 2.—*Computation of rainfall increments*

COMPUTATION OF STATION WEIGHTS

Station (1)	Rainfall over Thiessen polygon (2)	Percent of basin area (3)	Rainfall at station (4)	Weight, col. (2) x col. (3) / 100 x col. (4) (5)
A	4.3	38.9	4.73	0.35
B	4.6	37.0	5.56	.31
C	2.8	21.1	2.06	.29
D	5.0	3.0	4.06	.04

COMPUTATION OF WEIGHTED AVERAGE HOURLY RAINFALL OVER BASIN

Time, hours	Station A			Station B			Station C			Station D			Weighted average, sum of cols. (3)
	Mass rf. (1)	Δ rf. (2)	0.35xΔrf. (3)	Mass rf. (1)	Δ rf. (2)	0.31xΔrf. (3)	Mass rf. (1)	Δ rf. (2)	0.29xΔrf. (3)	Mass rf. (1)	Δ rf. (2)	0.04xΔrf (3)	
0				0									
1				.17	0.17	0.053				0			0.053
2	0			.33	.16	050	0			.15	0.15	0.006	056
3	20	0.20	0.070	.52	.19	059	09	0.09	0.026	.29	.14	006	161
4	.40	.20	070	.80	.28	087	17	08	.023	.52	23	009	189
5	73	.33	116	1.20	.40	.124	32	15	044	.84	32	013	297
6	1.20	47	.164	1.41	21	065	.52	20	.058	1.01	.17	007	294
7	1.20	0	0	1.85	.44	136	52	0	0	1.34	.33	013	149
8	2.05	85	.298	2.91	1.06	329	.89	.37	.107	2.05	.71	.028	.762
9	2.80	75	262	3.49	58	.180	1.22	.33	.096	2.47	42	017	555
10	3.15	.35	.122	4.19	70	217	1.37	15	.044	3.00	.53	021	404
11	3.90	.75	.262	4.79	.60	.186	1.70	.33	.096	3.40	.40	016	.560
12	4.20	.30	.105	5.08	.29	.090	1.83	.13	.038	3.63	.23	009	242
13	4.40	.20	.070	5.18	.10	.031	1.92	.09	.026	3.73	.10	004	131
14	4.40	0	0	5.18	0	0	1.92	0	0	3.83	.10	004	.004
15	4.59	19	.066	5.49	31	.096	2.00	.08	.023	3.97	.14	.006	191
16	4.70	.11	.038	5.56	.07	.022	2.04	.04	.012	4.04	.07	.003	075
17	4.73	.03	.010	5.56	0	0	2.06	.02	.006	4.06	02	001	.017
Total		4.73	1.653		5.56	1.725		2.06	.599		4.06	.163	4.140

The first step is to construct the Thiessen polygons, which are the areas bounded by the perpendicular bisectors of lines joining adjacent precipitation stations. The percentage of the drainage area controlled by each station's pologon is planimetered and entered in table 2. Next, the average depth of rainfall over each station's polygon is determined by planimetering areas between isohyets on figure 8(B). A factor to be used in weighing station rainfall values is obtained by multiplying the percentage of the drainage area controlled by each station's polygon by the ratio of the average depth of rainfall over each station's polygon to the observed rainfall at the station and dividing by 100.

Hourly incremental rainfall values are determined for each precipitation station from the mass curves of figure 8(A) and are multiplied by the appropriate weight factors as shown in table 2, to obtain the total for the drainage area.

Additional information on determining average rainfall is given in "Cooperative Studies Technical Paper No. 1," published by U.S. Weather Bureau.

(4) *Determination of rainfall excess.*—Two methods may be used to determine rainfall excess: By assuming a constant average retention rate throughout the storm period, and by assuming a retention rate varying with time. The capacity rate of retention decreases progressively throughout the storm period until a constant minimum rate is reached if the rain is sufficiently prolonged. With dry antecedent conditions, the initial capacity rate will be greater and will decline faster. Because the use of a varying retention rate requires a complicated method of computation, and because present knowledge of the exact shape of the infiltration curve is rather limited, it is often preferable to assume an average retention rate (sometimes referred to as infiltration index) with an estimate of initial loss being made if antecedent conditions are relatively dry.

The method of determining the period of rain-

fall excess, when an average retention rate is used, is a trial-and-error process in which a retention rate is assumed and subtracted from hourly rainfall increments determined as the average over the basin. Various retention rates are assumed until the total of the computed rainfall excess equals the measured storm runoff. An example of this procedure is given in table 3. If the correct retention rate has not been assumed after two trials, a rainfall excess-retention curve will facilitate the solution. In the example of table 3, the curve could be drawn through the two points represented by the coordinates 0.25, 1.37, and 0.15, 2.15. The correct retention rate corresponding to a rainfall excess of 2.0 inches would then be taken from this curve.

TABLE 3.—*Computation of rainfall excess*

Time, hours	Rainfall increment (basin average), inches	First trial		Second trial		Third trial	
		Assumed retention rate, inches per hour	Rainfall excess, inches	Assumed retention rate, inches per hour	Rainfall excess, inches	Assumed retention rate, inches per hour	Rainfall excess, inches
0							
1	0.05	0.25		0.15		0.17	
2	.06						
3	.16				0.01		
4	.19				.04		0.02
5	.30		0.05		.15		.13
6	.29		.04		.14		.12
7	.15		0		0		0
8	.76		.51		.61		.59
9	.56		.31		.41		.39
10	.40		.15		.25		.23
11	.56		.31		.41		.39
12	.24		0		.09		.07
13	.13				0		0
14	0				0		0
15	.19				.04		.02
16	.08						
17	.02	.25		.15		.17	
Total	4.14		1.37		2.15		1.96

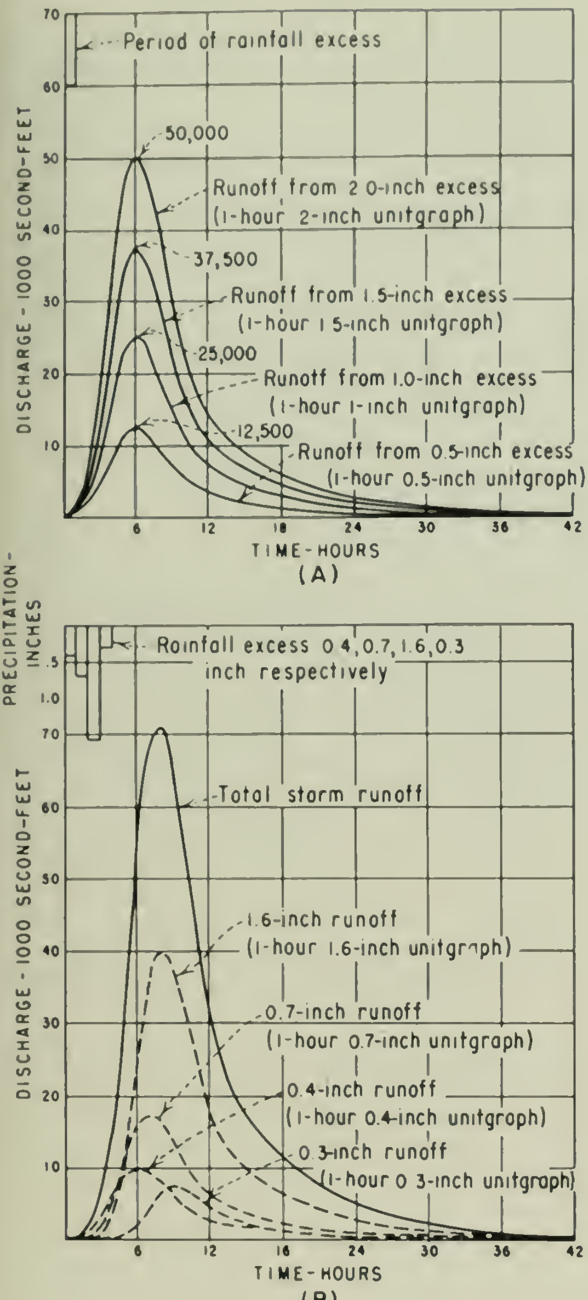
Total rainfall, 4.14 inches; observed runoff, 2.0 inches; total retention in 17 hours, 2.1 inches. The average retention rate of 0.17 inches per hour assumed in the third trial gives the best agreement of computed rainfall excess with measured runoff.

The duration time of excess rainfall is that time during which rainfall increments exceed the average retention rate. In the third trial, table 3, the duration time is 9 hours.

(c) *Estimating Direct Runoff from Soil and Cover Data.*—The problem most often encountered in flood studies is the need for estimating runoff from a watershed for which there are no records of runoff or precipitation. An approach to solution of this problem is to compare runoff characteristics of the watershed under study with those of watersheds for which records are available. Basin characteristics which may be most readily compared for estimating the volume of runoff that will result from a given amount of rainfall are soil type and cover which includes land usage. (The distribution of the runoff, i.e., the unitgraph, is compared on the basis of area, channel characteristics, etc.) Obviously, a great deal of judgment must be used in making such comparisons because of the great number of soil types and cover types and the many combinations thereof possible within a specific watershed. However, near minimum loss rates are associated with inflow design floods; therefore, only data on minimum loss rates for given soils and cover combinations need be determined.

Hydrologists of the Soil Conservation Service constantly encounter the problem of estimating direct runoff where no records are available for the specific watershed. As a result, a general procedure for estimating direct runoff has been developed to meet Soil Conservation Service needs. Discussions relating to this procedure taken from the "Hydrology Guide for Use in Watershed Planning," published by the Soil Conservation Service (see bibliography, sec. 55) are given in appendix A. A feature of the Soil Conservation Service method that should prove particularly useful is the classification of about 2,000 soils of the United States into four hydrologic soil groups representing a progression in degree of runoff potential from high to low. Although the Soil Conservation Service procedure for estimating direct runoff has been selected for use in this text due to its simplicity, one modification has been introduced in adapting this procedure to inflow design flood computations.

The runoff equation used in the Soil Conservation Service procedure does not provide for inclusion of a minimum retention rate after soil saturation. The runoff curves, figures A–4 (appendix A) represent solutions of the runoff equation that result in runoff increments almost equal to precipitation increments after the fifth or sixth hour of many of the design storm values obtained



Definitions:

Unitgraph - A hydrograph of storm runoff at a given point that will result from an isolated event of rainfall excess occurring within a unit of time and spread in an average pattern over the contributing drainage area. Identified by by the unit time and volume of the excess rainfall, that is 1-hour 1-inch unitgraph.

Rainfall excess - That portion of rainfall that enters a stream channel as storm runoff and produces the runoff hydrograph at the measuring point.

Basic Assumptions:

(1) The effects of all physical characteristics of a given drainage basin are reflected in the shape of the storm runoff hydrograph for that basin.

(2) At a given point on a stream, discharge ordinates of different unitgraphs of the same unit time of rainfall excess are mutually proportional to respective volumes. See (A) at left.

(3) A hydrograph of storm discharge that would result from a series of bursts of excess rain or from continuous excess rain of variable intensity may be constructed from a series of over lapping unitgraphs each resulting from a single increment of excess rain of unit duration. See (B) at left.

Practical Application:

For a given runoff contributing area, a unitgraph representing exactly one inch of runoff (rainfall excess) for a selected unit time interval is computed. Increments of rainfall excess for the same unit time interval are determined for a storm. A total hydrograph of runoff from the storm is then computed using assumptions (2) and (3) above. See graph (B) at left.

Figure 9. Unitgraph principles.

from data presented in sections 45 and 53. Infiltrometer studies indicate that all but impervious clay soils have a minimum constant infiltration rate after saturation that may range from about 0.05 inch per hour to greater than 1.00 inch per hour, depending on the type of soil. A progressively higher minimum rate should be associated with each of the four hydrologic soil groups arranged from D to A; however, data are not available at this time to propose firm rates. It is suggested that the following minimum retention rates for soil groups other than class D be used: For group A soils, 0.10 inch per hour; and for groups B and C soils, 0.05 inch per hour.

An example of how the data given in appendix A can be applied to inflow design flood studies is given in section 53.

47. *Unitgraph Principles.*—The basic tool for hydrograph computation is the unitgraph. Its fundamental principles are presented in abbreviated form on figure 9.

48. *Hydrograph Analysis.*—A procedure of hydrograph analysis is presented on figure 10. Storm duration and distribution over a watershed affect the shape of the resulting unitgraphs. Direct averaging of unitgraphs of different storm durations gives erroneous results. However, such unitgraphs can be averaged by converting the unitgraphs to dimensionless form as shown on figure 10(B).

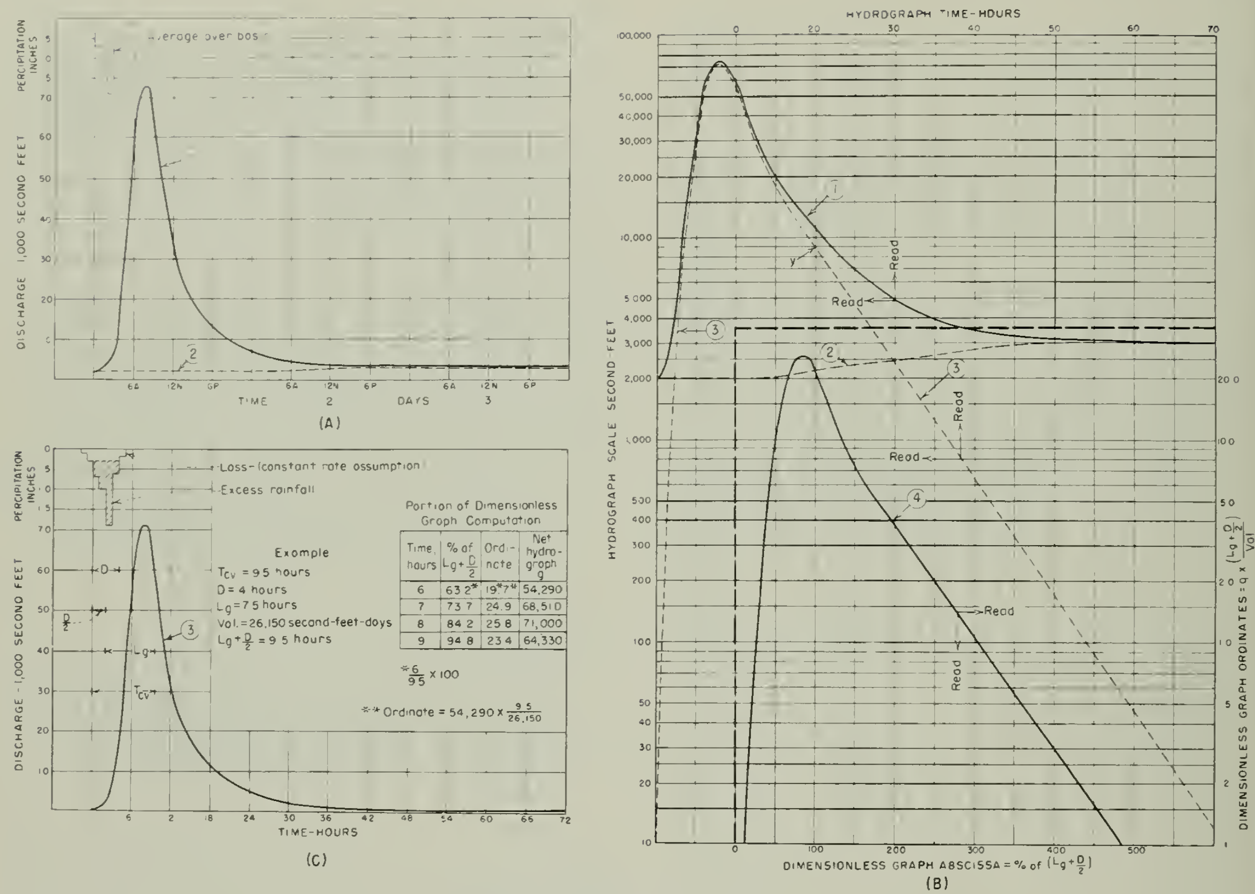


Figure 10. **Hydrograph analysis.**

HYDROGRAPH ANALYSIS (Refer to Fig. 10)

Given: Recorded hydrograph at given point on a stream. Rainfall data may or may not be available.

Required: Factors for deriving unitgraph to be applied at point of derivation, at another point on stream if of comparable runoff characteristics, or to comparable ungaged watershed.

Procedures:

(a) Plot recorded hydrograph on cartesian coordinate paper and on semilog paper:
① on figure 10(A), and
① on figure 10(B).

(b) Estimate base flow, ② on figure 10 (A) and (B), by trial and error. Subtract base flow from recorded hydrograph and plot net hydrograph, ③ on figure 10(B). If the base flow has been estimated correctly, the descending limb of hydrograph ③ on figure 10(B) will be a straight line (exponential recession). (③=① minus ② on figure 10(B).)

(c) Compute volume of net hydrograph ③ as follows:

(1) Add average hourly discharges (in sec.-ft.-hours) to a point such as y on the exponential recession, ③ on figure 10(B).

(2) Compute hourly recession constant, k_{hr}, from two points on exponential recession line by use of following equation:

$$k_{hr}=\sqrt[t]{\frac{q_t}{q_o}}$$

where:
q_o=discharge at first point,
q_t=discharge at second point, and
t=time interval, in hours, between points 1 and 2.

(3) Storage, or volume after y in (sec.-ft.-hours) equals:

$$\frac{-q_y}{\log_e k_{hr}}$$

where:

q_y = discharge in second-feet at point y, and

$\log_e k_{hr} = 2.3026\ (\log_{10} k_{hr})$.

(4) Total volume is sum of volume to y plus volume after y.

(d) For comparison with rainfall data, convert volume of ③ to inches of runoff:

$$\text{Inches of runoff} = \frac{\text{volume in sec.-ft.-hours}}{(\text{area in sq. mi.}) \times 645.3}$$

(e) Analyze rainfall data, if available; determine period D of rainfall excess.

(f) Compute time of occurrence of one-half volume of hydrograph ③, figure 10(C). The time to center of volume, T_{cr}, equals time from beginning of rise of net hydrograph to time one-half volume has passed measuring point.

(g) Find lag, L_g, time in hours from midpoint of excess rainfall period to time of occurrence of one-half volume.

(h) Compute dimensionless graph as follows and plot on semilog paper, ④ on figure 10(B).

(1) Abscissa—hours from beginning of excess rain expressed as percent of $(L_g + D/2)$. When rainfall data are not available $L_g + D/2$ may be taken equal to T_{cr}.

(2) Ordinates—discharge in second-feet of ③ (at respective abscissa) multiplied by $(L_g + D/2)$, all divided by net hydrograph volume expressed as sec.-ft.-days $\left(\frac{\text{sec.-ft.-hours}}{24}\right)$.

49. *Unitgraph Derivation for Ungaged Areas.*—An example of the derivation of a unitgraph for an ungaged area is shown on figure 11. Since hydrographs recorded at a dam site are seldom available, a means of transferring unitgraphs to ungaged areas is necessary. The factor "lag time" is used for this purpose. Lag time is an index to the time of concentration of runoff from a basin. It may be determined from recorded hydrographs and empirically correlated with basin characteristics. As shown on figure 11, determination of a lag time for an ungaged watershed from basin measurements provides the key to unitgraph computation for that watershed if a representative dimensionless graph is known. There are several definitions of lag time, each pertaining to a specific form of unitgraph derivation; two different definitions are used in this text. Lag time is defined as (1) the time from the center of excess rainfall to the time of occurrence of one-half the volume of the hydrograph—used with the dimensionless graph procedure outlined on figures 10 and 11; and (2) the time from the center of excess rainfall to the time of peak discharge of the hydrograph—used with the triangular hydrograph concept, figure 12.

UNITGRAPH DERIVATION FOR UNGAGED AREA (Refer to Fig. 11)

Given: Dimensionless graph (C) and lag relationship curve (B) for comparable area.

Required: Unitgraph for basin above given point.

Procedure:

(a) Outline drainage boundary, determine area (fig. 11(A)).

(b) Find:

L = length of longest watercourse from point of interest to watershed divide, measured in miles.

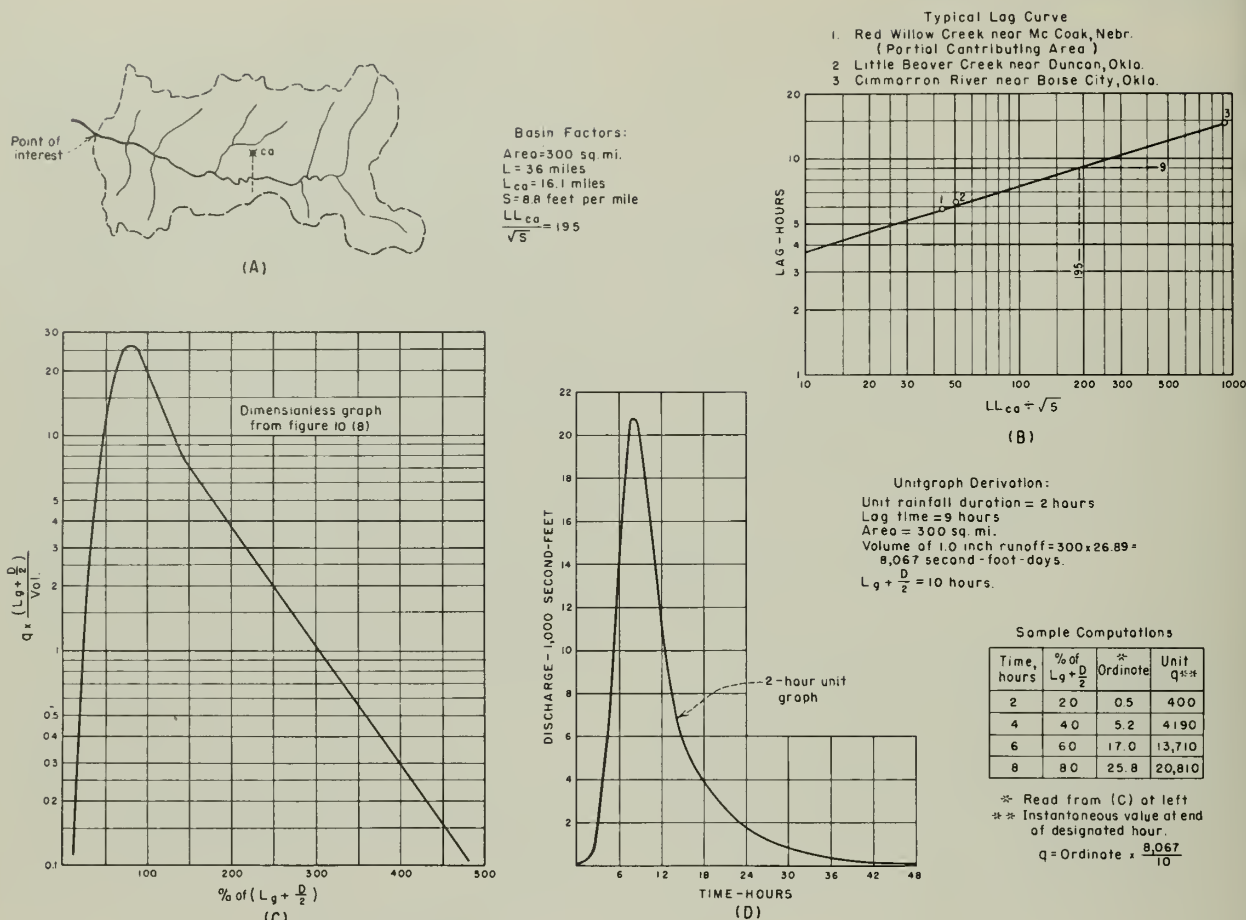
ca = centroid of basin—usually found by vertically suspending a cardboard cutout of basin shape successively from two or more points and finding intersection of plumb lines from each point.

L_{ca} = length of watercourse from point of interest to intersection of perpendicular from ca to stream alinement.

S = overall slope in feet per mile of longest watercourse from point of interest to divide.

(c) Solve for factor:

$$\frac{LL_{ca}}{\sqrt{S}}$$



Time, hours	% of $L_g+\frac{D}{2}$	* Ordinate	Unit q**
2	20	0.5	400
4	40	5.2	4190
6	60	17.0	13,710
8	80	25.8	20,810

* Read from (C) at left
** Instantaneous value at end of designated hour.
$q = \text{Ordinate} \times \frac{8{,}067}{10}$

***Figure 11.* Unitgraph derivation for ungaged area.**

(d) Enter graph, figure 11(B), with $\frac{LL_{ca}}{\sqrt{S}}$ value and read the corresponding lag time. (Points establishing empirical curves such as figure 11(B) are constructed by plotting lag times obtained from hydrograph analyses (fig. 10) versus respective $\frac{LL_{ca}}{\sqrt{S}}$ factors for basins of similar runoff characteristics.)

(e) Select a dimensionless graph (fig. 11(C))—usually the mean dimensionless graph of a number of dimensionless graphs derived for the same stream or for streams of similar characteristics.

(f) Select unit rainfall duration time—should be one-fourth or less of lag time for basin.

(g) Compute unitgraph (fig. 11(D)) using:
  (1) Basin area.
  (2) Lag time plus one-half unit rainfall duration time.
  (3) Dimensionless graph.

(h) Computed unitgraph may be used as shown on figure 9(B).

50. *Triangular Hydrograph Analysis.*—The practicability of representing a hydrograph as a triangle and the relationships that can be developed by such representation is presented on figure 12.

51. *Estimating Time of Concentration.*—As shown on figure 12, the general equation for computing the peak discharge resulting from a given amount of runoff is $q_p = \frac{484AQ}{T_p}$ in which $T_p = \frac{D}{2} + 0.6T_c$. The term $0.6T_c$ is an empirical relationship adopted by hydrologists of the Soil Conservation Service as representative of L, lag time, which is defined as the time in hours from the midpoint of excess rainfall, D, to the time of peak discharge. The

relationship of L to other hydrograph factors is shown on figure 12(B). Computation of hydrographs for ungaged watersheds by application of the equations given on figure 12 is dependent on an estimate of T_c, time of concentration of the watershed, defined as the travel time of water from the hydraulically most distant point of the watershed to the point of interest. Various methods of estimating T_c are given on figure 13. These data are admittedly generalized because the concentration of runoff is affected by storm distribution over the watershed and storm intensity as well as by the watershed's hydraulic characteristics. If data are available, more than one method of estimating time of concentration should be used to select a representative value for a particular watershed.

52. *Application of Triangular Hydrographs.*—Examples of the application of the triangular hydrograph concept are presented on figure 14.

53. *A Method of Computing an Inflow Design Flood.*—The following procedure is based to a large extent on that given in the Soil Conservation Service "Hydrology Guide for Use in Watershed Planning," [6] but with modifications to meet the purpose of this text. The resulting inflow design flood hydrograph represents direct runoff from precipitation in the form of rain over a watershed having no unusual runoff characteristics. If a certain amount of flow is expected to be in the channel at the time an inflow design flood could occur, the amount of this base flow should be added to the inflow design flood hydrograph. The procedure outlined assumes an ungaged area. Computations for a gaged area will differ only in that data regarding unitgraph characteristics and retention losses obtained from analyses of recorded runoff should be used.

(a) *Watershed Data.*—The following watershed data are always needed:

(1) Geographical location.

(2) Map showing topography, streams and drainage area.

(3) Information about soils and vegetative cover and their distribution throughout the watershed.

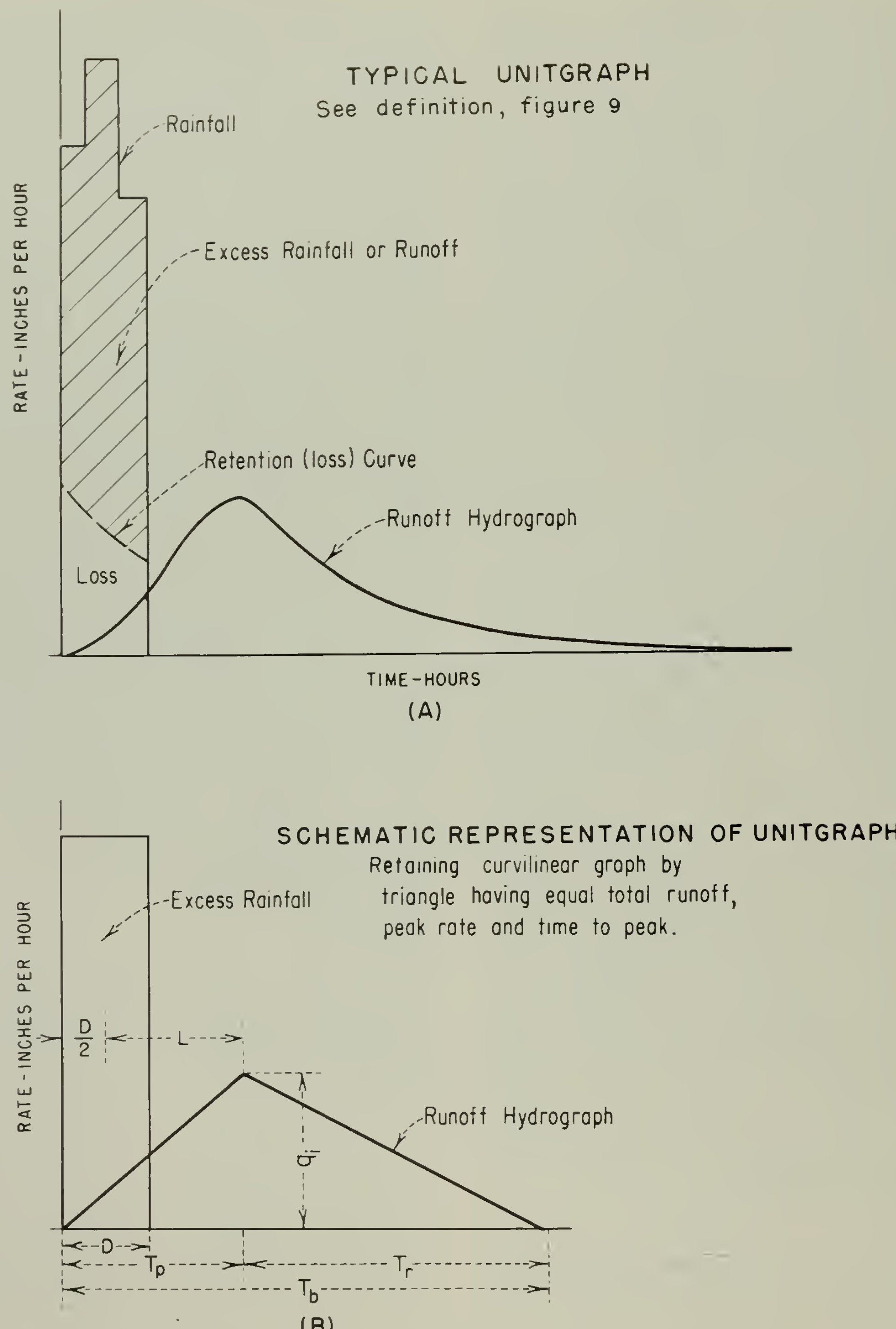
[6] See bibliography, sec. 55.

(b) *Magnitude Criteria.* It has been pointed out in section 40 that an inflow design flood may not always be equivalent to the maximum probable flood. In those instances where it is permissible to design for less than the maximum probable flood, a reasonable reduction in the severity of the factors affecting flood potential can be made. In this text, conditions designated as assumption A and assumption B to be used when considering design floods less than the maximum probable are given to provide a method for achieving a consistent degree of magnitude as related to the risk of damage which would result from structure failure. The same design storm values for a specific watershed are used for both assumptions A and B. These storm values are less than probable maximum; in most instances, they represent precipitation amounts slightly greater than those which have been observed for respective locations within the United States. The method of obtaining these values is given in section 45.

For assumption A, runoff from the design storm is computed assuming antecedent moisture condition III which presupposes the watershed soils to be nearly saturated (see appendix A). For assumption B, runoff from the design storm is computed assuming antecedent moisture condition II which presupposes the watershed soils to be less wet than condition III and comparable to the average condition at the time of occurrence of the maximum annual flood.

The following criteria for design floods are thus established:

If	*Then*
(1) Failure of structure would result in probable loss of human life.	(1) Inflow design flood is equivalent to the maximum probable flood.
(2) Failure would cause great damage to property and project operation but loss of human life is not envisioned.	(2) Inflow design flood may be as much less than the maximum probable as that obtained by assumption A.
(3) Failure would cause only loss of structure with little additional damage to property and project operation.	(3) Inflow design flood may be as much less than the maximum probable as that obtained by assumption B.



Source. Paper by Victor Mockus, Hydraulic Engineer, Soil Conservation Service, Central Technical Unit Beltsville, Md., February, 1957

Figure 12. Triangular hydrograph analysis. (Sheet 1 of 2.)

PEAK EQUATION DEVELOPMENT

Using triangle from (B) at left

$$Q = \frac{q_1 T_p}{2} + \frac{q_1 T_r}{2}$$

$$q_1 = \frac{2Q}{T_p + T_r}$$

Let $T_r = HT_p$, where H is a constant to be determined for a particular watershed

$$q_1 = \frac{2}{(1+H)} \frac{Q}{T_p}$$

Convert inches per hour to second-feet, introduce drainage area, A in square miles (1 inch per hour = 645.3 second-feet per square mile.)

$$q_p = \frac{2(645.3)}{(1+H)} \frac{AQ}{T_p}$$

or

$$q_p = \frac{KAQ}{T_p}$$

where

$$K = \frac{1290.6}{1+H}$$

EXPLANATION

Q = Total runoff in inches

q_1 = Peak rate, inches per hour

T_p = Time in hours from start of rise to peak rate

T_r = Time in hours from peak rate to end of triangle

q_p = Peak rate in second-feet

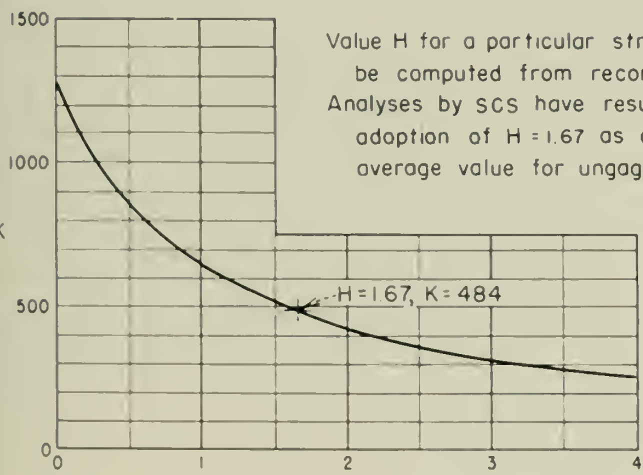
D = Rainfall excess period, hours

L = Lag, time from center of excess rainfall to time of peak, hours

T_c = Time of concentration-travel time of water from hydraulically most distant point to point of interest

T_b = Time base of hydrograph

Empirical relationship for lag
$L = 0.6\, T_c$



$H = T_r / T_p$

(C)

Value H for a particular stream may be computed from recorded hydrographs. Analyses by SCS have resulted in their adoption of H = 1.67 as a general average value for ungaged watersheds

General Peak Equation:

$$q_p = \frac{484 AQ}{T_p}$$

For $H = 1.67$
$T_b = 2.67\, T_p$

or using $L = 0.6\, T_c$

$$q_p = \frac{484 AQ}{D/2 + 0.6\, T_c}$$

since

$$T_p = \frac{D}{2} + 0.6\, T_c$$

Figure 12. Triangular hydrograph analysis (Sheet 2 of 2)

Purpose: A time of concentration from which a lag time can be computed must be obtained for hydrograph construction representing runoff from a watershed. Various methods of estimating time of concentration, T_C, are as follows:

A. ESTIMATING T_c FROM STREAM HYDRAULICS (SCS GUIDE)

1. Obtain stream reaches and channel cross-sections from field surveys.
2. Find approximate channel bankfull discharge for each reach.
3. Compute average velocity for the bankfull discharge of each reach.
4. Use the average velocity and the valley length of the reach to compute travel time through each reach.
5. Add travel times of reaches to get T_C.

Note: Appendix B "Hydraulic Computations" presents methods of computing flows in natural channels.

B. ESTIMATING T_c FROM VELOCITY ESTIMATES AND WATERCOURSE LENGTHS

Velocity Estimate Guide

U.S. Navy - Technical Publication Navdocks TP-PW-5 Table 8B, March 1953

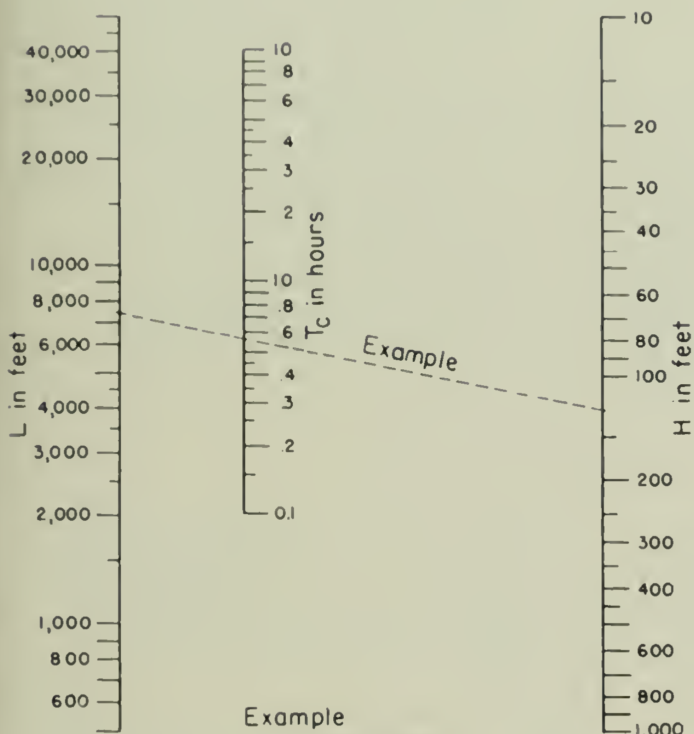
Average slope of channel from farthest point to outlet, in percent	Average velocity, feet per second
1 to 2	2.0
2 to 4	3.0
4 to 6	4.0
6 to 10	5.0

Texas Highway Department Rational Design of Culverts and Bridges, October 1946

Slope in percent	Average velocity, feet per second		
	Woodlands (upper portion watershed)	Pastures (upper portion watershed)	Natural channel not well defined
0 – 3	1.0	1.5	1.0
4 – 7	2.0	3.0	3.0
8 – 11	3.0	4.0	5.0
12 – 15	3.5	4.5	8.0

Figure 13. Time of concentration estimates. (Sheet 1 of 2.)

C. ESTIMATING T_c FROM LENGTHS AND SLOPES:



Example
L = 7,250 feet
H = 130 feet
then T_c = 0.57 hours

(a) Nomograph (SCS Guide)

L = length of longest water-course in feet
H = difference in elevation in feet between outlet point and divide

(b.) Solution may be made by equation from California Culverts Practice, California Highways and Public Works, September 1942.

$$T = \left(\frac{11.9\,L^3}{H}\right)^{0.385}$$

$T = T_c$ in hours
L = length of longest watercourse in miles
H = elevation difference in feet

Lag, L, (SCS Guide) may be estimated directly for a basin by subdividing into tributary drainage subareas and using the relationship:

$$L = \frac{\Sigma a_x T_x}{A}$$, where

L = lag in hours
a_x = the x-th increment of area in sq. mi.
T_x = travel time in hours from center of a_x to main basin outlet
A = total area of basin, sq. mi.

Figure 13. Time of concentration estimates. (Sheet 2 of 2.)

Simple Triangular Hydrograph:

Given:

A = 8.0 sq. mi.
T_c = 3.0 hours
D = 2.0 hours
Q = 1.0 inch

Required: Triangular hydrograph (Needed: T_p, q_p, T_b)

Computations: (Ref.– Figure 12)

$$T_p = \frac{D}{2} + 0.6\,T_c = \frac{2}{2} + 0.6\,(3.0)$$

$$T_p = 2.8 \text{ hours}$$

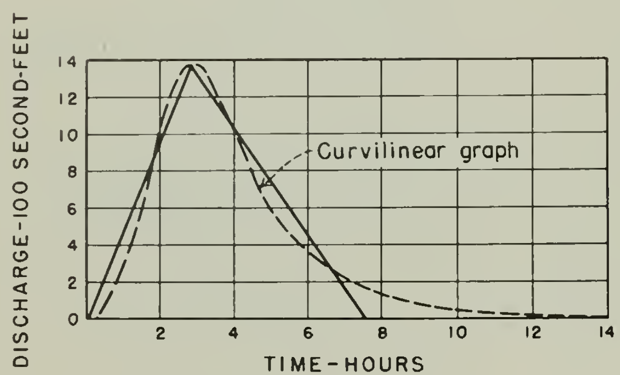
$$q_p = \frac{484\,A\,Q}{T_p} = \frac{(484)(8.0)(1.0)}{2.8}$$

$$q_p = 1{,}380 \text{ second-feet}$$

$$T_b = 2.67\,T_p$$

$$T_b = 7.48 \text{ use } 7.5 \text{ hours}$$

Plot T_p, T_b. Connect origin and T_p, T_p and T_b by straight lines.



(A)

A curvilinear hydrograph may be constructed from values of q_p and T_p by using ratios tabulated below which were obtained by SCS analyses of many unitgraphs

Time ratio, T/T_p	Disch. ratio, q/q_p	Example–see (A) above	
		Time, hours	Discharge, second-feet
0	0	0	0
0.1	0.015	0.28	21
0.2	0.075	0.56	104
0.3	0.16	0.84	220
0.4	0.28	1.12	386
0.5	0.43	1.40	596
0.6	0.60	1.68	828
0.7	0.77	1.96	1,063
0.8	0.89	2.24	1,228
0.9	0.97	2.52	1,339
1.0	1.00	2.8	1,380
1.1	0.98	3.08	1,352
1.2	0.92	3.36	1,270
1.3	0.84	3.64	1,159
1.4	0.75	3.92	1,035

Time ratio, T/T_p	Disch. ratio, q/q_p	Example–see (A) above	
		Time, hours	Discharge, second-feet
1.5	0.66	4.20	911
1.6	0.56	4.48	773
1.8	0.42	5.04	580
2.0	0.32	5.60	442
2.2	0.24	6.16	331
2.4	0.18	6.72	248
2.6	0.13	7.28	179
2.8	0.098	7.84	135
3.0	0.075	8.40	104
3.5	0.036	9.80	50
4.0	0.018	11.20	25
4.5	0.009	12.60	12
5.0	0.004	14.00	5
Infinity	0		

Figure 14. Application of triangular hydrograph. (U.S. Soil Conservation Service.) (Sheet 1 of 2.)

Complex Hydrograph Due To Variable Rainfall Increments:

Given: Area = 100 sq. mi.

T_c = 10 hours

Storm of D = 6 hours, successive 2 – hour increments, ΔD, of 0.6, 1.4, 0.8 inches, rainfall excess.

Procedure: For variable intensity storms, total storm is subdivided into duration increments of time equal to 1/5 (or less) of T_c. In this example, ΔD equals 2 hours

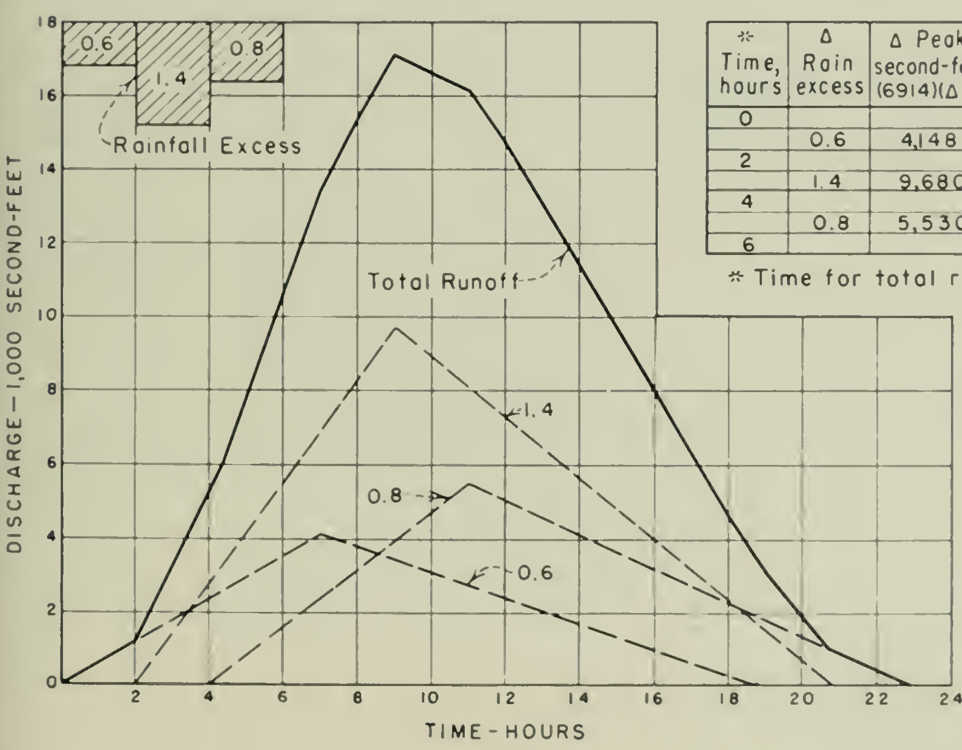
Computation: Compute simple hydrograph values for T_p, T_b and q_p for 1.0 inch of rainfall excess (runoff) occuring in a 2-hour period (1/5 T_c).

In example, T_p = 7.0 hours, T_b = 18.7 hours, q_p = 6,914 second-feet† for 1.0-inch runoff, 2 hours. For ΔD = 2 hours, T_p and T_b remain constant, q_p varies as the ratio of Δ rainfall excess to 1.0-inch excess. Prepare plotting table, plot simple hydrographs for each ΔD, add ordinates of simple hydrographs to get total runoff.

PLOTTING TABLE

* Time, hours	Δ Rain excess	Δ Peak, second-feet (6914)(ΔQ)	Δ Hydrograph Begin time	T of peak	T of end
0					
	0.6	4,148	0	7	18.7
2					
	1.4	9,680	2	9	20.7
4					
	0.8	5,530	4	11	22.7
6					

* Time for total runoff.



(B)

† Ref. Soil Conservation Service Hydrology Guide.

Figure 14. Application of triangular hydrograph. (U.S. Soil Conservation Service) (Sheet 2 of 2)

(c) *Computation of Inflow Design Floods—East of 105° Meridian.*—This subsection illustrates the procedure for computing the three floods for drainage areas east of the 105° meridian (examples 1, 2, and 3), and subsection (d) illustrates the procedure for drainage areas west of the 105° meridian.

Example 1.—Maximum Probable Flood

General procedure

1. Determine geographical location and size of drainage area.
2. Determine zone number and 6-hour 10-square-mile probable maximum precipitation from fig. 1.
3. Determine the design storm rainfall increments as follows:
   (a) Adjust the 6-hour 10-square-mile rainfall to values for the given drainage area size and to longer durations by use of fig. 2.
   (b) Determine hourly amounts of rainfall within maximum (first) 6-hour period by percentage values of curve for zone C, fig. 4.
   (c) Tabulate total 48-hour storm sequence by incremental time periods showing incremental rainfall amounts and accumulative amounts. The incremental rainfall during the maximum 6-hour period should be rearranged from the descending order of magnitude obtained above to the following hourly magnitude sequence, 1 through 6 hours: 6, 4, 3, 1, 2, 5. This is a judicial arrangement that gives a computed flood greater than one based on the assumption that the greatest hourly increment of rain occurs during the first hour of a storm and a smaller flood than that computed by assuming the greatest hourly increment of rain occurs during the 6th hour of a storm. The hourly increments of a storm cannot be predicted.

Specific application

1. For this example use a site in Peoria County, Ill. Drainage area is 61 square miles.
2. Zone 7:
   6-hour 10-square-mile probable maximum precipitation is 25.5 inches.
3. (a) Using fig. 2, obtain maximum probable precipitation for area of 61 square miles for various durations:

Duration, hours	Percent of 10-square-mile 6-hour value	Total rain, inches
0–6	83	21.2
0–12	100	25.5
0–24	111	28.3
0–48	121	30.9

(b) Hourly rainfall 6-hour period:

Time, hours	Percent, 6-hour rain	Accumulative rain, inches	Incremental rain, inches
1	49	10.4	10.4
2	64	13.6	3.2
3	75	15.9	2.3
4	84	17.8	1.9
5	92	19.5	1.7
6	100	21.2	1.7

(c) Design storm:

Time, hours	Incremental rain, inches	Accumulative rain, inches
0–1	1.7	1.7
1–2	1.9	3.6
2–3	2.3	5.9
2–3	10.4	16.3
4–5	3.2	19.5
5–6	1.7	21.2
6–12	4.3	25.5
12–24	2.8	28.3
24–48	2.6	30.9

Example 1.—Maximum Probable Flood—Continued.

General procedure	Specific application
4. Determine the hydrologic soil-cover complex number of the watershed (see appendix A). Availability of soil maps is discussed in sec. 82. If soil maps are not available, an estimate of the hydrologic soil-cover complex number can be made during the field inspection trip to the watershed.	4. Soil series indicated by maps are: Lindley, Clinton, Carrington, Grundy. Table A–1 (appendix A) shows the following hydrologic groups: Lindley—C, Clinton—B, Grundy—C, Carrington—B. Over 50 percent of the area is composed of either Clinton or Carrington soils; therefore the hydrologic group for the watershed is B. Land usage is estimated as follows, and applicable runoff curve numbers obtained from table A–2 (appendix A):

Percent	Usage	Curve No.
45	Row crops, contoured, good	75
35	Legumes, contoured, good	69
20	Pasture, contoured, good	35

Hydrologic soil-cover complex number equals:

$$0.45\times 75=33.75$$
$$0.35\times 69=24.15$$
$$0.20\times 35=\ \ 7.00$$
$$64.90$$

Curve number equals 65, condition II (fig. A–4, appendix A).

Note.—The above example is based on general soils map and estimated land usage. These data should be verified by field inspection in actual practice.

5. Estimate direct runoff, using runoff curve for hydrologic soil-cover complex number for condition II (see appendix A) and storm rainfall tabulated in step 3(e) above.

Antecedent moisture condition II is used for maximum probable flood computation east of the 105° meridian because of the extremely rare likelihood of an occurrence of a storm equivalent to probable maximum precipitation in conjunction with thoroughly wet soils. Condition II assumes soil moisture supply within the watershed to be similar to average conditions present before occurrence of the maximum annual flood.

Runoff curves, fig. A–4, do not contain the dimension time. Because limiting loss values discussed in sec. 46(c) are rates with time as a factor, application of these curves to design flood computations requires some modifications as follows:

Example 1.—Maximum Probable Flood—Continued.

General procedure

(a) Read appropriate curve, fig. A–4, using accumulated rainfall amounts by progressive time increments and determine accumulated direct runoff for respective progressive time increments.

(b) Compute and tabulate incremental rainfall and respective incremental runoff; subtract incremental runoff from incremental rainfall to determine incremental loss. (Loss computations beyond 12 hours are seldom necessary—see (c) below.)

(c) Rainfall-runoff curves, fig. A–4, give lower loss rates with increase in storm precipitation. When incremental loss rates determined by (b) above reach the limits stated in sec. 46(c), (0.10 inch per hour for group A soils; 0.05 inch per hour for groups B and C soils) the runoff curves are no longer used. The incremental runoff is then computed by subtracting the limiting loss rate amounts from the incremental rainfall.

Specific application

5. (a), (b), (c):

ESTIMATED DIRECT RUNOFF
[Values in inches]

Time, hours	Incremental rain [1]	Accumulative rain	Runoff: Accumulative	Runoff: Incremental	Incremental loss
0–1	1.7	1.7	[2] 0.08	0.08	1.62
1–2	1.9	3.6	.80	.72	1.18
2–3	2.3	5.9	2.28	1.48	.82
3–4	10.4	16.3	11.3	9.0	1.4
4–5	3.2	19.5	14.3	3.0	.2
5–6	1.7	21.2	15.9	1.6	.1
6–12	4.3	25.5	[3] 20.0	4.0	.30
12–24	2.8	28.3	----------	2.2	.60
24–48	2.6	30.9	----------	1.4	1.20

[1] From step 3(c).
[2] Estimate hundredths when scale permits.
[3] Curve 65 gives 20.0 inches runoff. 20.0 minus 15.9=4.1 inches incremental runoff. 4.3 minus 4.1 gives 0.2 inch loss in 6 hours or 0.03+ inch per hour. Abandon curve and use 0.05 inch per hour loss to compute runoff.

General procedure

6. Determine time of concentration for the watershed (sec. 51 and fig. 13).

Specific application

6. From map study: $L=18.5$ miles. Elevation difference between headwater and site, 400 feet=H. Since these are the only data available, T_c is estimated by solution of equation given on figure 13.

$$T_c=\left(\frac{11.9\ L^3}{H}\right)^{0.385}$$

$$=\left[\frac{(11.9)(18.5)^3}{400}\right]^{0.385}$$

$$=7.5 \text{ hours}$$

General procedure

7. Compute triangular hydrograph for each increment of runoff (figs. 12 and 14) as follows:

(a) Determine time increment, D, to be used. For the most intense period of the storm, the time increment, D, should be at least as small as one-fifth of the time of concentration.

(1) For the first (most intense) 6 hours, D will usually be 1 hour. For rapid concentration times (T_c less than 3 hours), one-half hour is the minimum practical time increment, D, recommended for the most intense periods. For less rapid concentration times, D for the most intense periods may be longer. For T_c values of 10 to 15 hours, a D of 2 hours is recommended. For T_c values of 15 to 30 hours, a D of 3 hours is recommended.

(2) The time period, D, may be lengthened in the later part of the storm to reduce computations. This will result in a poorer definition of the recession limb of the hydrograph, but it has little effect on the design.

Specific application

7. (a):

(1) For first 6 hours, use $D=1$ hour.

(2) For second 6 hours, use $D=6$ hours. For second 12 hours, use $D=12$ hours.

(3) Ignore runoff after first 24 hours as $T_c=7.5$ hours.

Example 1.—Maximum Probable Flood—Continued

General procedure	Specific application

(3) Runoff from the 24–48-hour period has little effect on design and, therefore, may be neglected. Runoff hydrographs for only the first 24-hour period of design storm need be computed for watersheds having a T_c of 24 hours or less.

(b) For each time interval, D, compute the time to peak, T_p, base time, T_b, and the peak discharge, q_p, for 1 inch of runoff. Use equations given on fig. 12.

(b) Given:

$$T_p=\frac{D}{2}+0.6T_c$$

$$T_b=2.67\ T_p$$

$$q_p=\frac{484\ A\ Q}{T_p}$$

$$T_c=7.5\text{ hours}$$

$$A=61\text{ sq. mi.}$$

$$Q=1.00\text{ inch}$$

For $D=1$ hour:

$$T_p=\frac{1}{2}+0.6\ (7.5)$$

$$=5\text{ hours}$$

$$T_b=2.67\ (5)$$

$$=13.4\text{ hours}$$

$$q_p=\frac{484\ (61)\ (1.00)}{5}$$

$$=5{,}900\text{ sec.-ft.}$$

For $D=6$ hours:

$$T_p=\frac{6}{2}+0.67\ (7.5)$$

$$=7.5\text{ hours}$$

$$T_b=2.67\ (7.5)$$

$$=20\text{ hours}$$

$$q_p=\frac{484\ (61)\ (1.00)}{7.5}$$

$$=3{,}940\text{ sec.-ft.}$$

For $D=12$ hours:

$$T_p=\frac{12}{2}+0.6\ (7.5)$$

$$=10.5\text{ hours}$$

$$T_b=2.67\ (10.5)$$

$$=28.0\text{ hours}$$

$$q_p=\frac{484\ (61)\ (1.00)}{10.5}$$

$$=2{,}810\text{ sec.-ft.}$$

(c) Compute the peaks of triangular hydrographs for each increment of runoff. These peaks are obtained by multiplying the peak discharge for 1 inch of runoff (computed in the previous step) by the increment of runoff.

(d) Prepare a plotting table showing the peaks, starting time, time of peak, and ending time of each incremental hydrograph.

(c) and (d):

PLOTTING TABLE

Time, hours	Incremental runoff, inches	q_p for 1.00 inch	q_p for incremental runoff	Incremental hydrographs: Begin time[1]	Peak time[1]	End time[1]
0–1	0.08	5,900	470	0	5	13.4
1–2	0.72	5,900	4,250	1	6	14.4
2–3	1.48	5,900	8,730	2	7	15.4
3–4	9.0	5,900	53,100	3	8	16.4
4–5	3.0	5,900	17,700	4	9	17.4
5–6	1.6	5,900	9,440	5	10	18.4
6–12	4.00	3,940	15,760	6	13.5	26.0
12–24	2.20	2,810	6,180	12	22.5	40.0

[1] Hours.
[2] q_p values—second-feet.

Example 1.—Maximum Probable Flood—Continued.

General procedure	Specific application
8. Plot maximum probable flood as follows: (a) Plot incremental triangular hydrographs on plain coordinate paper. (b) Add ordinates of plotted hydrograph of maximum probable flood. Ordinates need be added only at the times represented by the start, peak, and end of each incremental hydrograph.	8. See fig. 15.

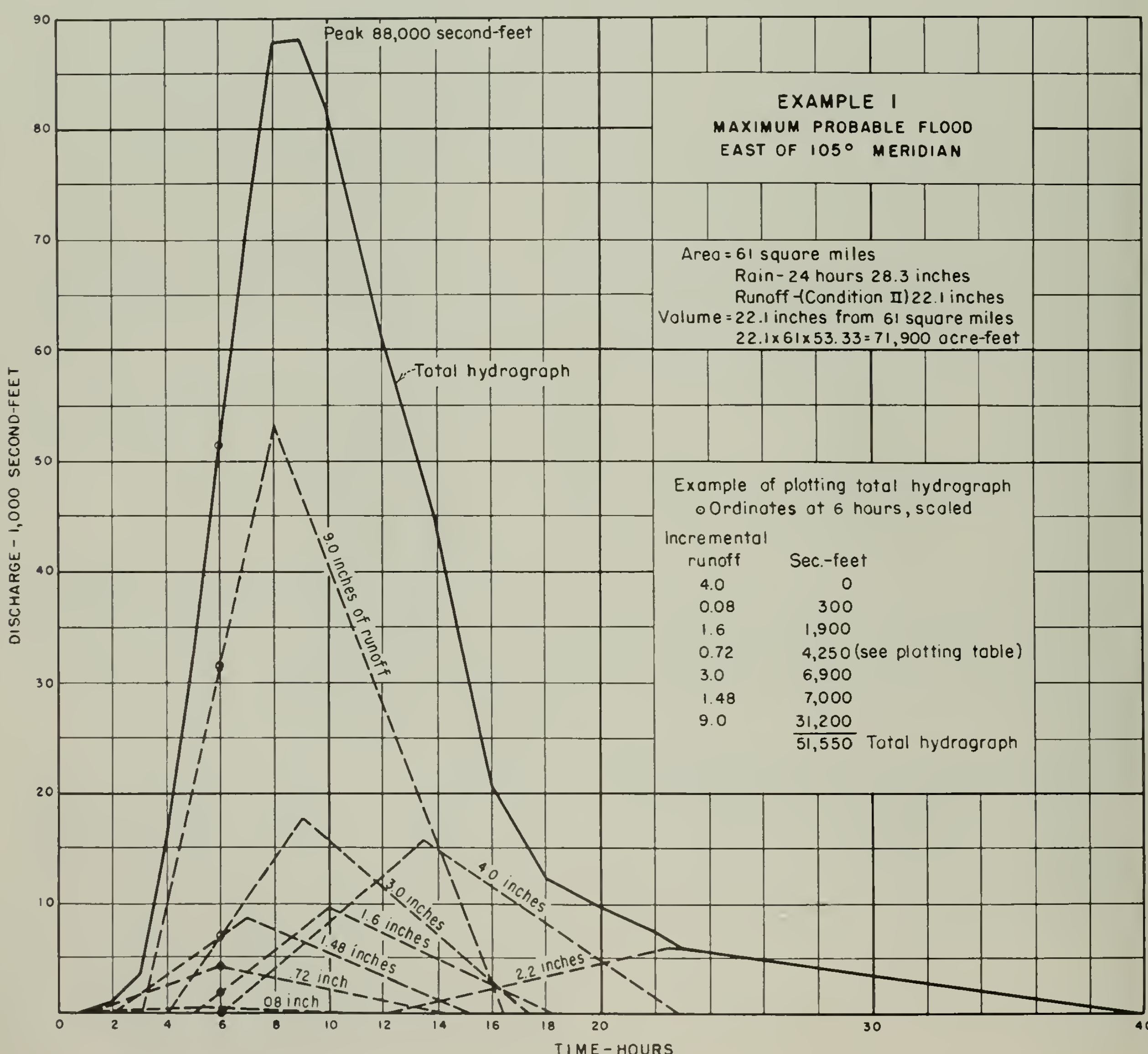


Figure 15. **Design flood—example 1.**

Example 2.—Flood Less Than the Maximum Probable—Assumption A

General procedure	Specific application

General procedure

1. Design storm values less than the probable maximum provide the basis for this flood. A rainfall reduction factor is obtained for geographical location from fig. 6. Apply this ratio directly to design storm values as arranged in step 3(c) of example 1, giving reduced design storm values. Distribution remains in same order.

Specific application

1. For Peoria County, Ill., factor equals 2.6. Reduction = 1/2.6, or 0.385.

REDUCED DESIGN STORM

Time, hours	Accumulative rain,[1] inches	Accumulative reduced rain, inches	Incremental reduced rain, inches
0-1	1.7	0.7	0.7
1-2	3.6	1.4	7
2-3	5.9	2.3	9
3-4	16.3	6.3	4.0
4-5	19.5	7.5	1.2
5-6	21.2	8.2	7
6-12	25.5	9.8	1.6
12-24	28.3	10.9	1.1
24-48	30.9	11.9	1.0

[1] From step 3(c), example 1.

General procedure

2. The hydrologic soil-cover complex will have been established for computation of a maximum probable flood (step 4 of example 1). Direct runoff from the reduced design storm is computed by the same procedure as outlined in step 5 of example 1 but using the runoff curve representing antecedent condition III moisture supply (which assumes watershed soils to be wet at the beginning of the design storm). See table A-5, curve conversions (appendix A).

Specific application

2. Curve 65, condition II, converts to curve 83, condition III. (See table A-5, appendix A.)

ESTIMATED DIRECT RUNOFF ASSUMPTION A
[Values in inches]

Time, hours	Incremental rain	Accumulative rain	Runoff: Accumulative	Runoff: Incremental	Incremental loss
0-1	0.7	0.7	0.02	0.02	0.68
1-2	7	1.4	.32	.30	.40
2-3	.9	2.3	90	58	.32
3-4	4.0	6.3	4.4	3.5	5
4-5	1.2	7.5	5.5	1.1	1
5-6	7	8.2	[1] 6.2	.65	.05
6-12	1.6	9.8		1.3	.30
12-24	1.1	10.9		.5	.60
24-48	1.0	11.9		0	[2] 1.20

[1] This value from curve gives zero loss for hour 5-6, therefore use 0.05 inch per hour loss for this period and subsequent periods.
[2] Total loss capacity; exceeds rainfall increment.

General procedure

3. The time of concentration for the watershed will be the same as that determined when computing the maximum probable flood (step 6 of example 1).

Specific application

3. $T_c = 7.5$ hours.

General procedure

4. Compute triangular hydrograph for each increment of runoff. The triangular unitgraphs for the time intervals, D, used for computing the maximum probable flood are applicable (step 7 of example 1).

Specific application

4.

PLOTTING TABLE—FLOOD ASSUMPTION A

Time, hours	Incremental runoff, inches	q_p for 1.00 inch	q_p for incremental runoff	Incremental hydrographs: Begin time [1]	Incremental hydrographs: Peak time [1]	Incremental hydrographs: End time [1]
0-1	0.02	5,900	120	0	5	13.4
1-2	30	5,900	1,770	1	6	14.4
2-3	58	5,900	3,420	2	7	15.4
3-4	3.5	5,900	20,650	3	8	16.4
4-5	1.1	5,900	6,490	4	9	17.4
5-6	65	5,900	3,840	5	10	18.4
6-12	1.3	3,940	5,120	6	13.5	26.0
12-24	5	2,810	1,400	12	22.5	40.0

[1] Hours.
q_p values—second-feet

General procedure

5. Plot the total flood hydrograph as discussed in step 8 of example 1.

Specific application

5. The plotted hydrograph is shown on fig. 16.

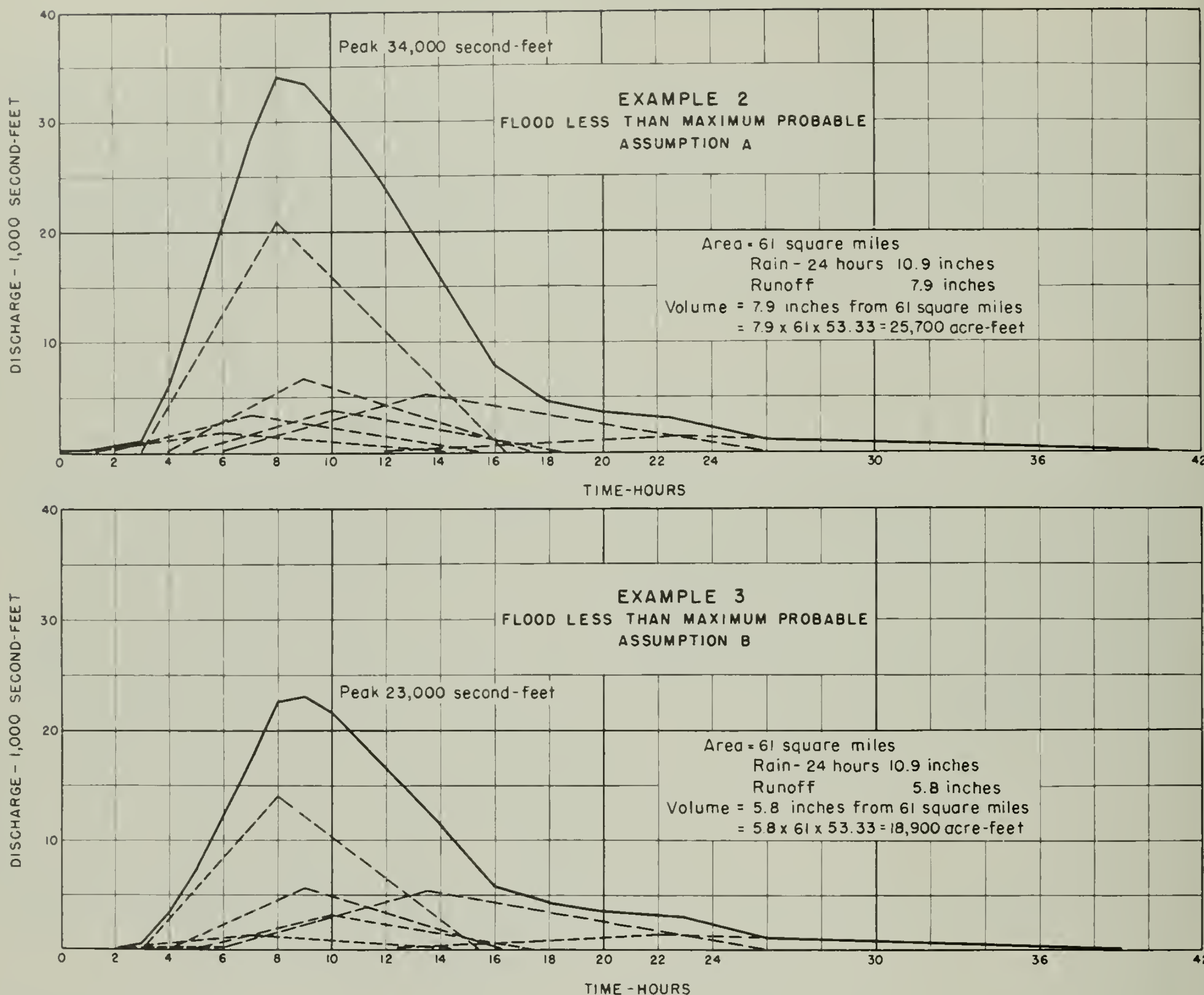


Figure 16. **Design flood—examples 2 and 3.**

Example 3.—Flood Less Than the Maximum Probable—Assumption B

1. The computational procedure is the same as previously outlined. Design storm values are the same as those for example 2. Direct runoff is computed assuming soil moisture of the watershed before the design storm begins to be represented by antecedent condition II (curve 65, see step 4 of example 1).
2. Computation of direct runoff and plotting data can be combined into one table as follows:

Time, hours	Incremental rain, inches [1]	Accumulative rain, inches [1]	Runoff, inches		Incremental loss, inches	q_p for 1.00 inch	q_p for incremental runoff	Incremental hydrographs		
			Accumulative [2]	Incremental				Begin time	Peak time	End time
0-1	0.7	0.7	0	0	0.7	....	0	0	0	0
1-2	.7	1.4	.02	.02	.68	5,900	120	1	6	14.4
2-3	.9	2.3	.22	.20	.68	5,900	1,200	2	7	15.4
3-4	4.0	6.3	2.58	2.36	1.64	5,900	13,920	3	8	16.4
4-5	1.2	7.5	3.50	.92	.28	5,900	5,430	4	9	17.4
5-6	.7	8.2	4.05	.55	.15	5,900	3,240	5	10	18.4
6-12	1.6	9.8	(5.4)	1.3	[3].30	3,940	5,120	6	13.5	26.0
12-24	1.1	10.9	....	.5	[3].60	2,810	1,400	12	22.5	40.0

[1] From step 1 of example 2. [2] From fig. A-4 (appendix A) for curve 65. [3] Use 0.05 inch per hour.

3. The plotted hydrograph is shown on figure 16.

(d) *Computation of Inflow Design Floods—West of 105° Meridian.* (1) *Maximum probable flood.*—The computational procedure is the same as previously described except that the following procedure is used for estimating the storm and the runoff:

Use figure 3 to obtain the 6-hour point rainfall values for the probable maximum design storm.

Use figure 5 to adjust the point storm values to values representing average rainfall over the given drainage area.

The design storm is extended for durations longer than 6 hours by the constants tabulated in table 1, section 45.

Hourly amounts of rainfall within the maximum 6-hour period are obtained from appropriate zonal curve, figure 4. The order of hourly distribution within the maximum 6-hour period is the same as given previously; that is, the hourly amounts in descending order from 1 through 6 hours obtained from a curve on figure 4 are rearranged in the following order of magnitude: 6, 4, 3, 1, 2, 5.

Direct runoff is computed assuming soil moisture supply preceding the design storm to be represented by antecedent condition III. This is a more severe runoff producing assumption than that assumed for similar conditions east of the 105° meridian. However, as discussed in section 45, design storm values west of the 105° meridian represent probable maximum storm values which are less severe than probable maximum precipitation values. The occurrence of a probable maximum storm when watershed soils are wet is a reasonable assumption when maximum storm values are used. The limiting retention rates given in section 46(c) are used.

(2) *Flood less than the maximum probable assumption A.*—The design storm reduction factor is obtained from figure 7. After applying this factor, runoff is estimated assuming antecedent condition III, and the hydrograph is computed as previously discussed.

(3) *Flood less than the maximum probable assumption B.*—Design storm values are the same as for assumption A. Runoff is estimated assuming antecedent condition II, and the hydrograph is computed as previously discussed.

TABLE 4.—*Logarithmic skew curve factors*[1]

[Multiply coefficient of variation by these and add to or subtract from the mean]

Coefficient of skew	Terms over mean percent	Percent of term above limit									Corresponding coefficient variation
		99 (−)	95 (−)	80 (−)	50 (−)	20 (+)	5 (+)	1 (+)	0.1 (+)	0.01 (+)	
0	50.0	2.32	1.64	0.84	0	0.84	1.64	2.32	3.09	3.72	0
0.1	49.4	2.25	1.62	.85	.02	.84	1.67	2.40	3.24	3.96	.03
0.2	48.7	2.18	1.59	.85	.03	.83	1.71	2.48	3.39	4.20	.06
0.3	48.1	2.12	1.56	.85	.05	.83	1.74	2.56	3.55	4.45	.10
0.4	47.5	2.05	1.53	.85	.06	.82	1.76	2.64	3.72	4.72	.13
0.5	46.9	1.99	1.50	.85	.08	.82	1.79	2.72	3.90	5.00	.16
0.6	46.3	1.92	1.47	.85	.09	.81	1.81	2.80	4.08	5.30	.20
0.7	45.6	1.86	1.44	.85	.11	.80	1.84	2.89	4.28	5.64	.23
0.8	45.0	1.80	1.41	.85	.12	.79	1.86	2.97	4.48	6.00	.27
0.9	44.4	1.73	1.38	.85	.14	.77	1.88	3.06	4.69	6.37	.30
1.0	43.7	1.68	1.31	.84	.15	.76	1.90	3.15	4.92	6.77	.33
1.1	43.1	1.62	1.31	.84	.17	.75	1.92	3.24	5.16	7.23	.37
1.2	42.5	1.56	1.28	.83	.18	.74	1.94	3.33	5.40	7.66	.41
1.3	41.9	1.51	1.25	.83	.19	.72	1.96	3.41	5.64	8.16	.44
1.4	41.3	1.46	1.22	.82	.20	.71	1.98	3.50	5.91	8.66	.48
1.5	40.7	1.41	1.19	.81	.22	.69	1.99	3.59	6.18	9.16	.51
1.6	40.1	1.37	1.16	.81	.23	.67	2.01	3.69	6.48	9.79	.55
1.7	39.5	1.32	1.13	.80	.24	.66	2.02	3.78	6.77	10.40	.59
1.8	38.9	1.27	1.10	.79	.25	.64	2.03	3.88	7.09	11.07	.62
1.9	38.3	1.23	1.07	.78	.26	.62	2.04	3.98	7.42	11.83	.66
2.0	37.7	1.19	1.05	.77	.27	.61	2.05	4.07	7.78	12.60	.70
2.1	37.1	1.15	1.02	.76	.28	.59	2.06	4.17	8.13	13.35	.74
2.2	36.5	1.11	.99	.75	.29	.57	2.07	4.27	8.54	14.30	.78
2.3	35.9	1.07	.96	.74	.30	.55	2.07	4.37	8.95	15.25	.82
2.4	35.3	1.03	.94	.73	.31	.53	2.08	4.48	9.35		.86
2.5	34.7	1.00	.91	.72	.31	.51	2.08	4.58	9.75		.90
2.6	34.1	.97	.89	.71	.32	.49	2.09	2.68	10.15		.94
2.7	33.5	.94	.86	.69	.33	.47	2.09	4.78	10.65		.98
2.8	32.9	.91	.84	.68	.33	.45	2.09	4.89	11.20		1.03
2.9	33.3	.87	.82	.67	.31	.43	2.09	5.01	11.75		1.08
3.0	31.8	.84	.79	.66	.31	.41	2.08	5.11	12.30		1.12
3.2	30.6	.78	.74	.64	.35	.37	2.06	5.35	13.50		1.22
3.4	29.4	.73	.69	.61	.35	.33	2.04	5.58			1.33
3.6	28.2	.67	.65	.58	.36	.28	2.02	5.80			1.44
3.8	27.0	.62	.61	.55	.36	.23	1.98	6.10			1.57
4.0	25.7	.58	.56	.52	.36	.19	1.95	6.50			1.70
4.5	22.2	.48	.47	.45	.35	.10	1.79	7.30			2.10
5.0	19.5	.40	.40	.39	.31	0	1.60	8.20			2.50

$$\text{Adjusted skew} = \text{Computed skew}\left(1+\frac{8.5}{n}\right).$$

The figures in the last column show the value of the coefficient of variation that, in connection with the coefficient of skew shown in the first column will produce plotting point of a line that is straight on log probability paper. The column "Terms over mean percent" represents the point of intersection of that straight line with the mean value of the variable.

[1] Reprinted by permission from A. Hazen's "Flood Flows."

54. *Frequency Curve Computations.*—Information shown on figures 17 and 18 gives the mathematical procedure for computing frequency curves by Hazen's method. Engineers basing decisions on results of flood frequency computations should keep in mind the points briefly discussed in section 42.

Probability Data for
Annual Peak Discharge
Ashley Creek near Vernal, Utah

Computed by ______
Date July 1944

Coefficient of Variation $= \sqrt{\dfrac{\Sigma \text{ Col. } 8}{n-1}} = \sqrt{\dfrac{4.18}{23}} = 0.43$

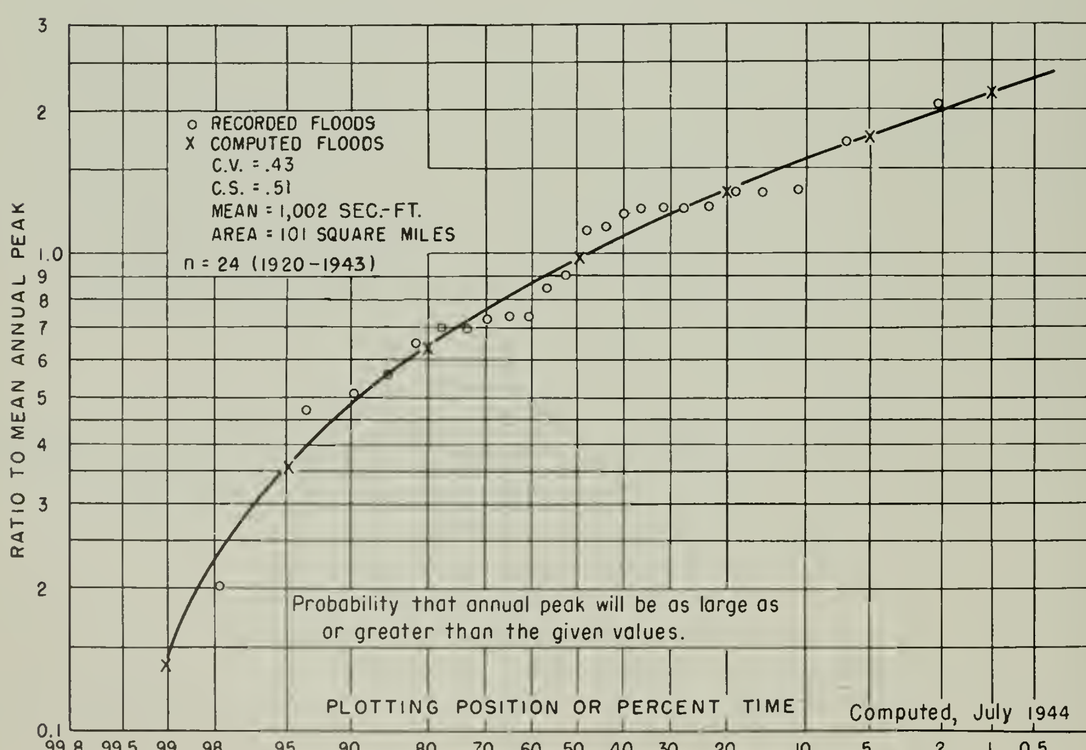
$F = 1 + \dfrac{8.5}{n} = 1.35$

Coefficient of Skew $= \dfrac{\Sigma \text{ Col. } 9 \times F}{(n-1)\,(c.v.)^3} = \dfrac{(0.69)\,(1.35)}{(23)\,(.080)} = 0.51$

1	2	3	4	5	6	7	8	9	10	11	12	13	14
Year	Items	Items in order of magnitude	Plotting position	Freq. in years	Ratio to mean	Col. 6 minus 1.0	Col. 7 squared	Col. 7 cubed	% of time	Skew factor CS=0.5	Col. 11 x C.V. (0.43)	Col. 12 plus 1.0	Col. 13 x mean
1920	1,360	2,050	2.08	48	2.05	1.05	1.10	1.16					
1921	2,050	1,700	6.25	16	1.70	.70	.49	.34	99	-1.99	-.86	.14	140
									95	-1.50	-.64	.36	361
	✓		✓			✓		✓	80	-.85	-.37	.63	631
1942	1,270	475	93.75	1.07	.47	-.53	.28	-.15	50	-.08	-.03	.97	972
1943	702	201	97.92	1.02	.20	-.80	.64	-.51	20	+.82	+.35	1.35	1,350
									5	+1.79	+.77	1.77	1,770
	Sum	24,038	Algebraic sum			+.03	4.18	+.69	1	+2.72	+1.17	2.17	2,170
Years of record n=24						Check							
	Mean	1,002				O.K.							

(A)

TABULATION AND COMPUTATIONS



(B)

CUMULATIVE FREQUENCY CURVE OF ANNUAL PEAKS
ASHLEY CREEK NEAR VERNAL, UTAH

Figure 17. Frequency curve computations—Hazen's method. (Sheet 1 of 2.)

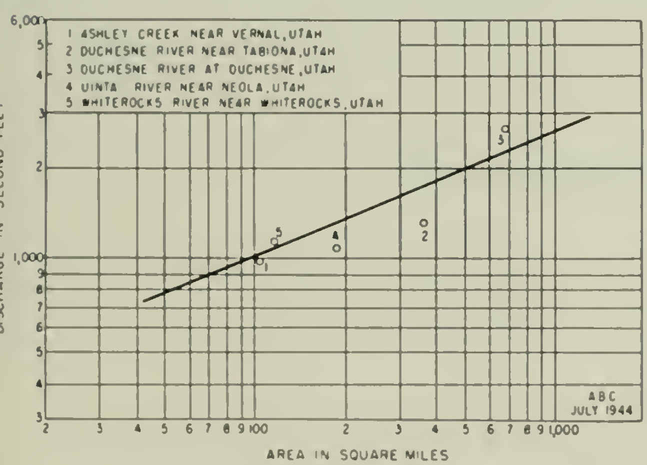
COMPUTATIONAL STEPS FOR FIGURE 17

1 Tabulate data in columns 1 and 2 If record is broken, try to fill gaps by correlation with nearby records If no correlations are possible, use available data as continuous record Consult other references for unusual cases such as one extreme event or incorporation of historical flows into current record
2 Column 3.- List items from column 2 in descending order of magnitude
3 Column 4. - See table 5.
4 Column 5.- 100 - column 4 values
5 Column 6.-Ratio of individual item in column 3 to arithmetical mean of items in column 3
6 Complete columns 7, 8, 9 by indicated computations
7 Compute Coefficient of Variation and Coefficient of Skew by equations shown in (A).
8 Fill in values column 11 from table 4.
9 Complete columns 12, 13, 14 by indicated computations
10 Columns 13 and 14 give values for a curve which should fit the data in columns 6 and 3 respectively Test of fit is obtained by plotting values column 4 vs column 6 and column 10 vs column 13 on logarithmic probability paper See (B) on left The same test of fit may be made by plotting on log-log paper the values in column 5 vs column 3, and drawing the curve through the points defined by values in column 10 divided into 100 vs column 14 values (This plotting not shown.)

Figure 17. Frequency curve computations—Hazen's method. (Sheet 2 of 2.)

No	Stream	Station	Years record	Record ending	Area, sq mi	Mean	c v	c s	n(c s)	n(c v)	Remarks
1	Ashley Creek	Vernal, Utah	24	1943	101	1,002	43	50	12 00	10 32	
2	Duchesne River	Tabiona, Utah	25	1943	352	1,357	40	20	5 00	10 00	
3	Duchesne River	Duchesne, Utah	26	1943	660	2,752	35	0	0	9 10	
4	Uinta River	Neola, Utah	17	1943	181	1,119	42	50	8 50	7 14	
5	Whiterocks River	Whiterocks, Utah	18	1943	115	1,162	59	1 00	18 00	10 62	
			110						43 50	47 18	
		Avg cs = $\frac{43.50}{110}$ = 396 use 40 Avg cv = $\frac{47.18}{110}$ = 429 use 43									A B C JULY 1944

(A) AVERAGE COEFFICIENTS OF VARIATION AND SKEW
(For Peak Discharges in Second-Feet)



(B) MEAN ANNUAL PEAK DISCHARGE - VERSUS DRAINAGE AREA

EXPLANATION

Given: Area, ungaged water shed (500 sq mi)
Records of comparable streams in vicinity
Required: Frequency curve of probable peak discharge for 500 sq mi.
Procedure:
1 Compute mean, c v and c s for each record See fig 17(A)
2 Tabulate data as shown in (A) and determine weighted average c v and c s as shown
3 Plot mean values vs respective drainage area on log-log paper as shown in (B)
4 Draw mean line through plotted points
5 Enter plot (B) with ungaged area (500), read mean (2,000).
6 Compute curve for ungaged area using
a Mean, 2,000 second-feet.
b Average c.s. and c.v. (see (A)).
c Procedure outlined by columns 10 through 14 of figure 17(A)

Figure 18. Flood frequency study—ungaged basin.

TABLE 5.—*Plotting points for probability paper for series ranging from 13 to 40 terms* [1]

TERMS IN RECORD																											
13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
3.85	3.57	3.33	3.13	2.94	2.78	2.63	2.50	2.38	2.27	2.17	2.08	2.00	1.92	1.85	1.79	1.72	1.67	1.61	1.56	1.52	1.47	1.43	1.39	1.35	1.32	1.28	1.25
11.54	10.71	10.00	9.38	8.82	8.33	7.89	7.50	7.14	6.82	6.52	6.25	6.00	5.77	5.56	5.36	5.17	5.00	4.84	4.69	4.55	4.41	4.29	4.17	4.05	3.95	3.85	3.75
19.23	17.86	16.67	15.62	14.71	13.89	13.16	12.50	11.90	11.36	10.87	10.42	10.00	9.62	9.26	8.93	8.62	8.33	8.06	7.81	7.58	7.35	7.14	6.94	6.76	6.58	6.41	6.25
26.92	25.00	23.33	21.87	20.59	19.44	18.42	17.50	16.67	15.91	15.22	14.58	14.00	13.46	12.96	12.50	12.07	11.67	11.29	10.94	10.61	10.29	10.00	9.72	9.46	9.21	8.97	8.75
34.62	32.14	30.00	28.12	26.47	25.00	23.68	22.50	21.43	20.45	19.57	18.75	18.00	17.31	16.67	16.07	15.52	15.00	14.52	14.06	13.64	13.24	12.86	12.50	12.16	11.84	11.54	11.25
42.31	39.29	36.67	34.37	32.35	30.56	28.95	27.50	26.19	25.00	23.91	22.92	22.00	21.15	20.37	19.64	18.97	18.33	17.74	17.19	16.67	16.18	15.71	15.28	14.86	14.47	14.10	13.75
50.00	46.43	43.33	40.62	38.24	36.11	34.21	32.50	30.95	29.55	28.26	27.08	26.00	25.00	24.07	23.21	22.41	21.67	20.97	20.31	19.70	19.12	18.57	18.06	17.57	17.11	16.67	16.25
57.69	53.57	50.00	46.87	44.12	41.67	39.47	37.50	35.71	34.09	32.61	31.25	30.00	28.85	27.78	26.79	25.86	25.00	24.19	23.44	22.73	22.06	21.43	20.83	20.27	19.74	19.23	18.75
65.38	60.71	56.67	53.12	50.00	47.22	44.74	42.50	40.48	38.64	36.96	35.42	34.00	32.69	31.48	30.36	29.31	28.33	27.42	26.56	25.76	25.00	24.29	23.61	22.97	22.37	21.79	21.25
73.08	67.86	63.33	59.37	55.88	52.78	50.00	47.50	45.24	43.18	41.30	39.58	38.00	36.54	35.19	33.93	32.76	31.67	30.65	29.69	28.79	27.94	27.14	26.39	25.68	25.00	24.36	23.75
80.77	75.00	70.00	65.62	61.76	58.33	55.26	52.50	50.00	47.73	45.65	43.75	42.00	40.38	38.89	37.50	36.21	35.00	33.87	32.81	31.82	30.88	30.00	29.17	28.38	27.63	26.92	26.25
88.46	82.14	76.67	71.87	67.65	63.89	60.53	57.50	54.76	52.27	50.00	47.92	46.00	44.23	42.59	41.07	39.66	38.33	37.10	35.94	34.85	33.82	32.86	31.94	30.08	30.26	29.49	28.75
96.15	89.29	83.33	78.12	73.53	69.44	65.79	62.50	59.52	56.82	54.35	52.08	50.00	48.08	46.30	44.64	43.10	41.67	40.32	39.06	37.88	36.76	35.71	34.72	33.78	32.89	32.05	31.25
	96.43	90.00	84.37	79.41	75.00	71.05	67.50	64.29	61.36	58.70	56.25	54.00	51.92	50.00	48.21	46.55	45.00	43.55	42.19	40.91	39.71	38.57	37.50	36.49	35.53	34.62	33.75
		96.67	90.62	85.29	80.56	76.32	72.50	69.05	65.91	63.04	60.42	58.00	55.77	53.70	51.79	50.00	48.33	46.77	45.31	43.94	42.65	41.43	40.28	39.19	38.16	37.18	36.25
			96.87	91.18	86.11	81.58	77.50	73.81	70.45	67.39	64.58	62.00	59.62	57.41	55.36	53.45	51.67	50.00	48.44	46.97	45.59	44.29	43.06	41.89	40.79	39.74	38.75
				97.06	91.67	86.84	82.50	78.57	75.00	71.74	68.75	66.00	63.46	61.11	58.93	56.90	55.00	53.23	51.56	50.00	48.53	47.14	45.83	44.59	43.42	42.31	41.25
					97.22	92.11	87.50	83.33	79.55	76.09	72.92	70.00	67.31	64.81	62.50	60.35	58.33	56.45	54.69	53.03	51.47	50.00	48.61	47.30	46.05	44.87	43.75
						97.37	92.50	88.10	84.09	80.43	77.08	74.00	71.15	68.52	66.07	63.79	61.67	59.68	57.81	56.06	54.41	52.86	51.39	50.00	48.68	47.44	46.25
							97.50	92.86	88.64	84.78	81.25	78.00	75.00	72.22	69.64	67.24	65.00	62.90	60.94	59.09	57.35	55.71	54.17	52.70	51.32	50.00	48.75
								97.62	93.18	89.13	85.42	82.00	78.85	75.93	73.21	70.69	68.33	66.13	64.06	62.12	60.29	58.57	56.94	55.41	53.95	52.56	51.25
									97.73	93.48	89.58	86.00	82.69	79.63	76.79	74.14	71.67	69.35	67.19	65.15	63.24	61.43	59.72	58.11	56.58	55.13	53.75
										97.83	93.75	90.00	86.54	83.33	80.36	77.59	75.00	72.58	70.31	68.18	66.18	64.29	62.50	60.81	59.21	57.69	56.25
											97.92	94.00	90.38	87.04	83.93	81.03	78.33	75.81	73.44	71.21	69.12	67.14	65.28	63.51	61.84	60.26	58.75
												98.00	94.23	90.74	87.50	84.48	81.67	79.03	76.56	74.24	72.06	70.00	68.06	66.22	64.47	62.82	61.25
													98.08	94.44	91.07	87.93	85.00	82.26	79.69	77.27	75.00	72.86	70.83	68.92	67.11	65.38	63.75
														98.15	94.64	91.38	88.33	85.48	82.81	80.30	77.94	75.71	73.61	71.62	69.74	67.95	66.25
															98.21	94.83	91.67	88.71	85.94	83.33	80.88	78.57	76.39	74.32	72.37	70.51	68.75
																98.28	95.00	91.94	89.06	86.36	83.82	81.43	79.17	77.03	75.00	73.08	71.25
																	98.33	95.16	92.19	89.39	86.76	84.29	81.94	79.73	77.63	75.64	73.75
																		98.39	95.31	92.42	89.71	87.14	84.72	82.43	80.26	78.21	76.25
																			98.44	95.45	92.65	90.00	87.50	85.14	82.89	80.77	78.75
																				98.48	95.59	92.86	90.28	87.84	85.53	83.33	81.25
																					98.53	95.71	93.06	90.54	88.16	85.90	83.75
																						98.57	95.83	93.24	90.79	88.46	86.25
																							98.61	95.95	93.42	91.03	88.75
																								98.65	96.05	93.59	91.25
																									98.68	96.15	93.75
																										98.72	96.25
																											98.75

[1] Reprinted by permission from A. Hazen's "Flood Flows."

C. SELECTED BIBLIOGRAPHY

55. *Bibliography.*

Creager, W. P., Justin, J. D., and Hinds, Julian, "Engineering for Dams," vol. I, John Wiley & Sons, Inc., New York, 1945.

Johnstone, Don, and Cross, W. P., "Elements of Applied Hydrology," The Ronald Press Co., New York, 1949.

Lindsey, R. K. Jr., Kohler, M. A., and Paulhus, J. L. H., "Applied Hydrology," McGraw-Hill Book Co., Inc., New York, 1949.

"Snow Hydrology—A Summary Report of the Snow Investigations," North Pacific Division, Corps of Engineers, U.S. Army, Portland, Oreg., June 30, 1956.

"Unitgraph Procedures," Bureau of Reclamation, Division of Project Investigations, Denver, Colo., 1952 (unpublished).

"Hydrology Guide for Use in Watershed Planning," National Engineering Handbook, Sec. 4, Supplement A, U.S. Department of Agriculture, Soil Conservation Service, Central Technical Unit, Beltsville, Md.

Hazen, A., "Flood Flows," John Wiley & Sons, Inc., New York, 1930.

Corbett, D. M., et al., "Stream Gaging Procedure," U.S. Geological Survey Water Supply Paper 888, 1943.

U.S. Weather Bureau Technical Papers

Paper No. 2, "Maximum Recorded United States Point Rainfall for 5 Minutes to 24 Hours at 207 First-Order Stations," A. L. Shands and D. Ammerman, Washington, D.C., April 1947.

Paper No. 4, "Topographically Adjusted Normal Iso-

hyetal Maps for Western Colorado," B. H. Russler and W. C. Spreen, Washington, D.C., August 1947.

Paper No. 5, "Highest Persisting Dewpoints in Western United States," Washington, D.C., January 1948.

Paper No. 14, "Tables of Precipitable Water and Other Factors for a Saturated Pseudo-Adiabatic Atmosphere," Washington, D.C., 1951.

Paper No. 15, "Maximum Station Precipitation for 1, 2, 3, 6, 12, and 24 Hours." Part I: Utah; Part II: Idaho, 1951; Part III: Florida, 1952; Part IV: Maryland, Delaware, and District of Columbia; Part V: New Jersey, 1953; Part VI: New England, 1953. Part VII: South Carolina, 1953. Part VIII: Virginia, 1954; Part IX: Georgia, 1954. Part X: New York, 1954; Part XI: North Carolina, 1955. Part XII: Oregon, 1955; Part XIII: Kentucky, 1955; Part XIV: Louisiana, 1955; Part XV: Alabama, 1955; Part XVI: Pennsylvania, 1956, Part XVII: Mississippi, 1956; Part XVIII: West Virginia, 1956. Part XIX: Tennessee, 1956; Part XX: Indiana, 1956.

Paper No. 16, "Maximum 24-Hour Precipitation in the United States," A. H. Jennings, Washington, D.C., January 1952.

Paper No. 19, "Mean Number of Thunderstorm Days in the United States," Washington, D.C., December 1952.

Paper No. 24, "Rainfall Intensities for Local Drainage Design in the United States" (For Durations of 5 to 240 Minutes and 2-, 5-, and 10-Year Return Periods), Part I: West of 115th Meridian, Washington, D.C., August 1953; Part II: Between 105° W. and 115° W., Washington, D.C., August 1954.

Paper No. 25, "Rainfall Intensity-Duration-Frequency Curves" (For Selected Stations in the United States, Alaska, Hawaiian Islands, and Puerto Rico), Washington D.C., December 1955.

Paper No. 28, "Rainfall Intensities for Local Drainage Design in Western United States" (For Durations of 20 Minutes to 24 Hours and 1- to 100-Year Return Periods), Washington, D.C., November 1956.

Paper No. 29, "Rainfall Intensity-Frequency Regime," Part 1—The Ohio Valley, Washington, D.C., June 1957.

Cooperative Studies Technical Papers, Cooperative Studies Section, Division of Climatological and Hydrologic Services, U.S. Weather Bureau, in cooperation with the Bureau of Reclamation

Paper No. 1, "Manual for Depth-Area-Duration Analysis of Storm Precipitation."

Cooperative Studies Reports, Cooperative Studies Section, Division of Climatological and Hydrologic Services, U.S. Weather Bureau, in cooperation with the Bureau of Reclamation

Report No. 9, "Maximum Possible Flood-Producing Meteorological Conditions." (1) Colorado River Basin above Glen Canyon Damsite, (2) Colorado River Basin above Bridge Canyon Damsite, (3) San Juan River Basin above Bluff Damsite, (4) Little Colorado River Basin above Coconino Damsite. June 1949.

Report No. 11, "Critical Meteorological Conditions for Design Floods in the Snake River Basin," February 1953.

Report No. 12, "Probable Maximum Precipitation on Sierra Slopes of the Central Valley of California," Washington, D.C., March 1954.

Hydrometeorological Reports, Hydrometeorological Section, Hydrologic Services Division, U.S. Weather Bureau, in cooperation with the U.S. Corps of Engineers

Report No. 2, "Maximum Possible Precipitation over the Ohio River Basin above Pittsburgh, Pennsylvania," 1942.

Report No. 3, "Maximum Possible Precipitation over the Sacramento Basin of California," 1943.

Report No. 5, "Thunderstorm Rainfall," 1947.

Report No. 15, "A Report on Depth-Frequency Relations of Thunderstorm Rainfall on the Sevier Basin, Utah," 1944.

Report No. 18, "Tentative Estimates of Maximum Possible Flood-Producing Meteorological Conditions in the Columbia River Basin," 1945.

Report No. 20, "An Estimate of Maximum Possible Flood-Producing Meteorological Conditions in the Missouri River Basin Above Garrison Dam Site," 1945.

Report No. 21, "A Hydrometeorological Study of the Los Angeles Area," 1939.

Report No. 21A, "Preliminary Report on Maximum Possible Precipitation, Los Angeles Area, California," 1944.

Report No. 21B, "Revised Report on Maximum Possible Precipitation, Los Angeles Area, California," 1945.

Report No. 22, "An Estimate of Maximum Possible Flood-Producing Meteorological Conditions in the Missouri River Basin Between Garrison and Fort Randall," 1946.

Report No. 23, "Generalized Estimates of Maximum Possible Precipitation Over the United States East of the 105th Meridian, for Areas of 10, 200, and 500 Square Miles," 1947.

Report No. 24, "Maximum Possible Precipitation Over the San Joaquin Basin, California," 1947.

Report No. 25, "Representative 12-Hour Dewpoints in Major United States Storms East of the Continental Divide," 1947.

Report No. 25A, "Representative 12-Hour Dewpoints in Major United States Storms East of the Continental Divide," 2d edition, 1949.

Report No. 28, "Generalized Estimate of Maximum Possible Precipitation Over New England and New York," 1952.

Report No. 33, "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1,000 Square Miles and Durations of 6, 12, 24, and 48 Hours," 1956.

Selection of Type of Dam

H. G. ARTHUR [1]

A. CLASSIFICATION OF TYPES

56. *General.*—Dams may be classified into a number of different categories, depending upon the purpose of the classification. For the purposes of this manual, it is convenient to consider three broad classifications according to: Use, hydraulic design, or materials comprising the structure.

57. *Classification According to Use.*—Dams may be classified according to the broad function which they are to serve, such as storage, diversion, or detention. Refinements of classification can also be made by considering specific functions involved.

Storage dams are constructed to impound water in periods of surplus supply for use in periods of deficient supply. These periods may be seasonal, annual, or longer. Many small dams impound the spring runoff for use in the summer dry season. Storage dams may be further classified according to the purpose of the storage, such as water supply, recreation, fish and wildlife, hydroelectric power generation, irrigation, etc. The specific purpose or purposes which are to be served by a storage dam often have an influence in the design of the structure, and may establish criteria such as the amount of reservoir fluctuation which may be expected and the amount of reservoir seepage which may be permitted. Figure 19 shows a small earthfill storage dam, and figure 20 shows a concrete-gravity structure serving both diversion and storage purposes.

Diversion dams are ordinarily constructed to provide head for carrying water into ditches, canals, or other conveyance systems to the place of use. They are used for irrigation developments, for diversion from a live stream to an off-channel-location storage reservoir, for municipal and industrial uses, or for any combination of the above. Figure 21 shows a typical small diversion dam.

Detention dams are constructed to retard flood runoff and minimize the effect of sudden floods. Detention dams fall into two main types. In one type, the water is temporarily stored ,and released through an outlet structure at a rate which will not exceed the carrying capacity of the channel downstream. In the other type, the water is held as long as possible and allowed to seep into pervious banks or gravel strata in the foundation. The latter type is sometimes called a water-spreading dam or dike because its main purpose is to recharge the underground water supply. Detention dams are also constructed to trap sediment. These often are called debris dams.

Although this is not as common on small projects as on large developments, often dams are constructed to serve more than one purpose. Where multiple purposes are involved, a reservoir allocation is usually made to each of the separate uses. A common multipurpose project involving small dams combines storage, flood control, and recreational uses.

58. *Classification by Hydraulic Design.*—Dams may also be classified as overflow or nonoverflow dams.

Overflow dams are designed to carry discharge over their crests. They must be made of materials which will not be eroded by such discharges. Concrete, masonry, steel, and wood are required excepting for overflow structures only a few feet high.

Nonoverflow dams are those which are not designed to be overtopped. This type of design

[1] Engineer, Earth Dams Section, Bureau of Reclamation

Figure 19. Crescent Lake Dam, a small earthfill storage dam on Crescent Creek in Oregon. 806–126–92.

extends the choice of materials to include earthfill and rockfill dams.

Often the two types are combined to form a composite structure consisting of, for example, an overflow concrete gravity dam with dikes of earthfill construction. Figure 22 shows a composite structure built by the Bureau of Reclamation.

59. *Classification by Materials.*—The most common classification used for purposes of discussion of design procedures is based upon the materials comprising the structure. This classification also usually recognizes the basic type of design such as, for example, concrete *gravity* dam or concrete *arch* dam.

This text is limited in scope to consideration of the more common type of small dams which are constructed under present day conditions; namely, *earthfill*, *rockfill*, and *concrete gravity*. Other types of dams, including concrete arch, concrete buttress, and timber dams, are discussed briefly, with an explanation of why their designs are not contained in this text.

60. *Earthfill.*—Earthfill dams are the most common type of dam, principally because their construction involves utilization of materials in the natural state requiring a minimum of processing. Moreover, the foundation requirements for earthfill dams are less stringent than for other types. It is likely that earthfill dams will continue to be more prevalent than other types for storage purposes, partly because the number of sites favorable for concrete structures is decreasing as a result of extensive water storage development, particularly in arid and semiarid regions where the conservation of water for irrigation is a fundamental necessity.

Although the earthfill classification includes several types, the development of modern exca-

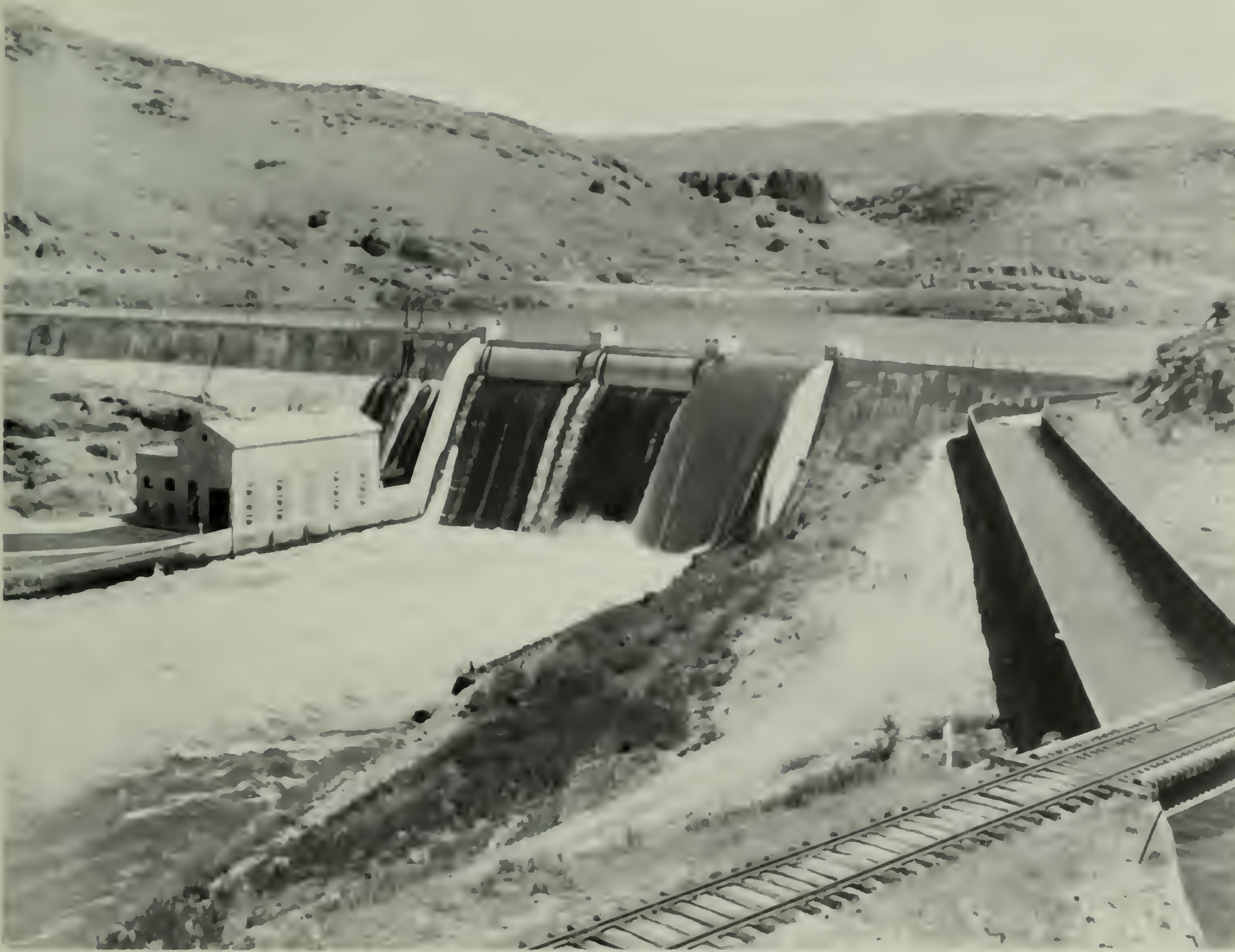


Figure 20. **Black Canyon Dam, a concrete-gravity storage and diversion structure on the Payette River in Idaho. Boise 2304.**

vating, hauling, and compacting equipment for earth materials has made the rolled-fill type so economical as to virtually replace the semi-hydraulic- and hydraulic-fill types of earthfill dams. This is especially true for the construction of small structures where the relatively small amount of material to be handled precludes the establishment of the large plant required for efficient hydraulic operations. For this reason, only the rolled-fill type of earthfill dam is treated in this text. Earthfill dams of the rolled-fill type are further classified as "homogeneous," "zoned," or "diaphragm," as described in chapter V.

Earthfill dams require supplementary structures to serve as spillways. The principal disadvantage of an earthfill dam is that it will be damaged or even may be destroyed under the erosive action of water flowing over it if sufficient spillway capacity is not provided. It is also subject to serious damage or even failure due to burrowing of animals unless special precautions are taken. Unless the site is offstream, provision must be made for diversion of the stream during construction through the damsite by means of a conduit, or around the damsite by means of a tunnel. Otherwise, special provisions, including the use of heavy rock sections, must be incorporated in the design to permit overflowing of the embankment during construction. This latter type of diversion should be attempted only by those experienced in this field.

61. ***Rockfill.*** The rockfill dam uses rock of all sizes to provide stability and an impervious membrane to provide watertightness. The membrane may be an upstream facing of impervious soil, a concrete slab, asphaltic concrete paving, steel plates, or another similar device; or it may be an interior thin core of impervious soil.

Like the earth embankments, the rockfill dam is

Figure 21. **Murdock Diversion Dam, a small diversion structure on the Provo River in northern Utah.**

subject to damage or destruction by the overflow of water and so must be provided with a spillway of adequate capacity to prevent overtopping of the dam. An exception is the extremely low diversion dam where the rockfill facing is designed specifically to withstand overflows. Rockfill dams require foundations which will not be subject to settlement of magnitudes sufficient to rupture the watertight membrane. The only suitable foundations, therefore, are rock or compact sand and gravel.

The rockfill type is adapted to remote locations where the supply of good rock is ample, where suitable soil for an earthfill dam is not available, and where the construction of a concrete dam would be too costly.

62. *Concrete Gravity.*—The concrete gravity dam is adapted to sites where there is a reasonably sound rock foundation, although low structures may be founded on alluvial foundations if adequate cutoffs are provided. It is well adapted for use as an overflow spillway crest and, because of this advantage, it is often used for the spillway feature of earthfill and rockfill dams or as the overflow section of a diversion dam.

In the early 1900's, some gravity dams were constructed of stone. However, the amount of hand labor required in this operation has been responsible for the exclusive use of concrete in modern gravity dam construction.

Gravity dams may be either straight or curved in plan. The curved plan may offer some advantage in both cost and safety. Also, occasionally the upstream curvature will locate that part of the dam on higher bedrock foundation.

63. *Concrete Arch.*—A concrete arch dam is adaptable to sites where the ratio of width between abutments to height is not great and where the foundation at the abutments is solid rock capable of resisting arch thrust. Because the design of an

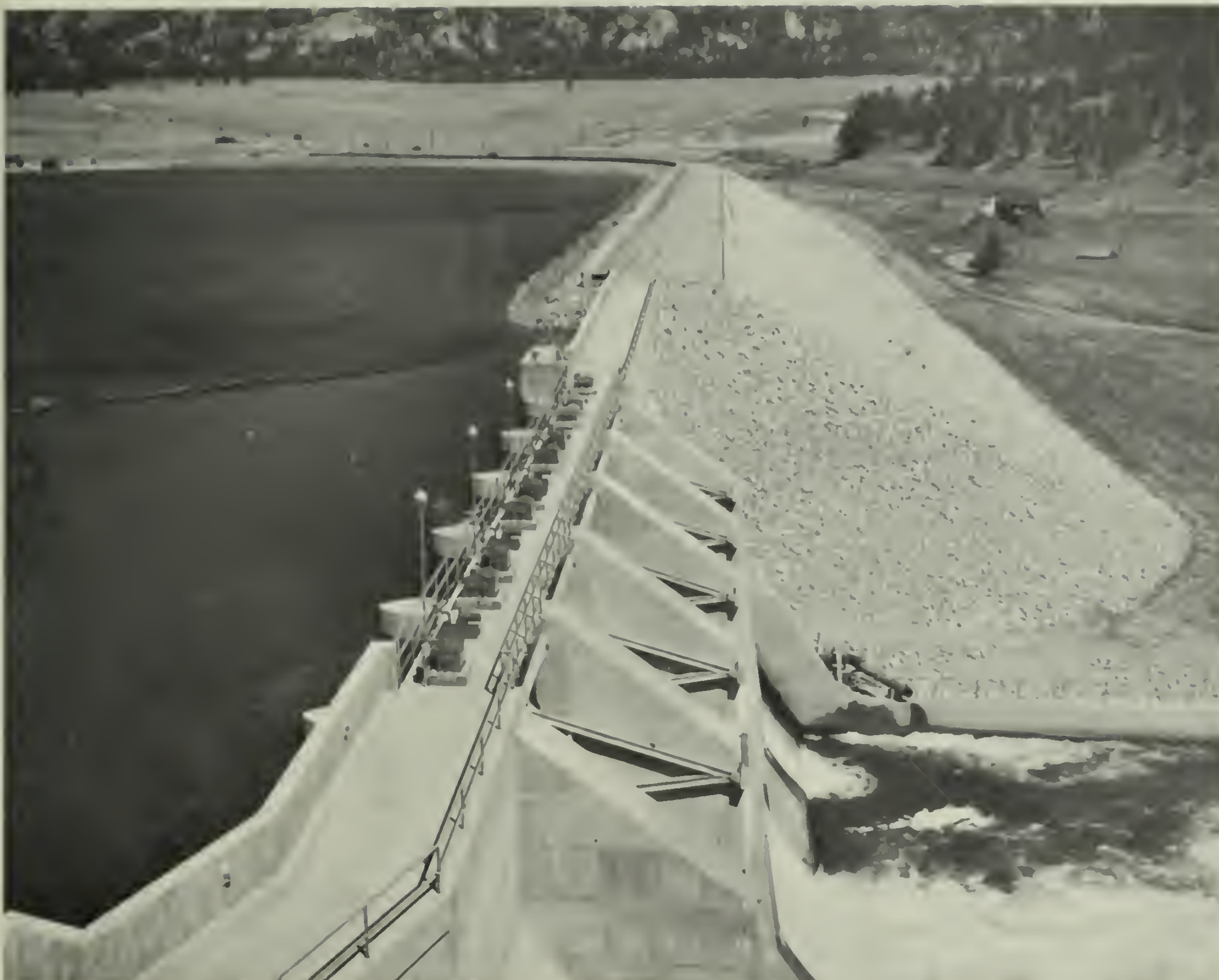


Figure 22. Olympus Dam, a combination earthfill and concrete-gravity structure on the Big Thompson River in Colorado. The concrete section contains the spillway and an outlet works to a canal. 245–704–3117.

arch dam is quite specialized, a detailed discussion of this type of design is not included in this text.

64 *Concrete Buttress.* Buttress dams comprise flat deck and multiple arch structures. They require about 60 percent less concrete than solid gravity dams, but the increased formwork and reinforcement steel required usually offset the savings in concrete. A number of buttress dams were built in the 1930's when the ratio of labor cost to materials cost was comparatively low. This type of construction usually is not competitive with other types of dams when labor costs are high.

The design of buttress dams is based on the knowledge and judgment that comes only from specialized experience in that field. Because of this fact and because of the limited application for the construction of buttress dams under present day conditions, the design of this type of structure is not included in this text.

65. *Other Types.* Dams of other types than those mentioned above have been built, but in most cases they meet some unusual local requirement or are of an experimental nature. In a few instances, structural steel has been used both for the deck and for the supporting framework of dams. Prior to 1920, a number of timber dams were constructed, particularly in the Northwest. The amount of labor involved in the timber dam, coupled with the short life of the structure, makes this type of structure uneconomical for modern construction. This and other uncommon types are not treated in this text.

B. PHYSICAL FACTORS GOVERNING SELECTION OF TYPE

66. *General.*—It is only in exceptional circumstances that an experienced designer can say that only one type of dam is suitable or most economical for a given damsite. Except in cases where the selection of type is obvious, it will be found that preliminary designs and estimates will be required for several types of dams before one can be shown to be most economical. It is, therefore, important to emphasize that the project is likely to be unduly expensive unless decisions regarding selection of type are based upon adequate study and after consultation with competent engineers.

In the selection of type for important structures, it is also usually advisable to secure the advice of an experienced engineering geologist in connection with the relative applicability of possible types to the foundation at the site.

In numerous cases, excessive cost of protection from spillway discharges, limitations of outlet works, and the problem of diverting the stream during construction have an important bearing on the selection of type. In certain cases, the selection of type may also depend upon the availability of labor and equipment. These may be particularly important considerations when the element of time is involved. Inaccessibility of site may also have an important bearing on the selection.

The selection of the best type of dam for a particular site calls for thorough consideration of the characteristics of each type, as related to the physical features of the site and the adaptation to the purposes the dam is supposed to serve, as well as economy, safety, and other pertinent limitations. The final choice of type of dam will generally be made after consideration of those factors. Usually, the greatest single factor determining the final choice of type of dam will be the cost of construction. The following paragraphs discuss important physical factors in the choice of type of dam.

67. *Topography.*—Topography, in large measure, dictates the first choice of type of dam. A narrow stream flowing between high, rocky walls would naturally suggest a concrete overflow dam. The low, rolling plains country would, with equal fitness, suggest an earthfill dam with a separate spillway. For intermediate conditions, other considerations take on more importance, but the general principle of satisfactory conformity to natural conditions is a safe primary guide.

The location of the spillway is an important item that will be governed very largely by the local topography and will, in turn, have a material bearing on the final selection of type of dam.

68. *Geology and Foundation Conditions.*—Foundation conditions depend upon the geological character and thickness of the strata which are to carry the weight of the dam, their inclination, permeability, and relation to underlying strata, existing faults, and fissures. The foundation will limit the choice of type to a certain extent, although such limitations will frequently be modified, considering the height of the proposed dam. The different foundations commonly encountered are discussed below:

(1) *Solid rock foundations*, because of relatively high bearing power and resistance to erosion and percolation, offer few restrictions as to the type of dam that can be built upon them. Economy of materials or overall cost will be the ruling factor. The removal of disintegrated rock together with the sealing of seams and fractures by grouting will frequently be necessary.

(2) *Gravel foundations*, if well compacted, are suitable for earthfill, rockfill, and low concrete gravity dams. As gravel foundations are frequently subject to water percolation at high rates, special precautions must be taken to provide effective water cutoffs or seals.

(3) *Silt or fine sand foundations* can be used for the support of low concrete gravity dams and earthfill dams if properly designed, but they are not suitable for rockfill dams. The main problems are settlement, the prevention of piping, excessive percolation losses, and protection of the foundation at the downstream toe from erosion.

(4) *Clay foundations* can be used for the support of earthfill dams but require special treatment. Since there may be considerable settlement of the dam if the clay is unconsolidated and the moisture content is high, clay foundations ordinarily are not suitable for the construction of concrete gravity dams, and should not be used for rockfill dams.

Tests of the foundation material in its natural state are usually required to determine the consolidation characteristics of the material and its ability to support the superimposed load.

(5) *Nonuniform foundations.* Occasionally, situations may occur where reasonably uniform foundations of any of the foregoing descriptions cannot be found and where a nonuniform foundation of rock and soft material must be used if the dam is to be built. Such unsatisfactory conditions can often be overcome by special design features. Each site, however, presents a problem for appropriate treatment by experienced engineers, and no attempt is made in this text to treat such unusual problems.

The details of the foundation treatments mentioned above are given in the appropriate chapters on the design of earthfill, rockfill, and concrete gravity dams.

69. *Materials Available.* Materials for dams of various types, which may sometimes be available at or near the site, are:

(1) Soils for embankments.

(2) Rock for embankments and riprap.

(3) Concrete aggregate (sand, gravel, crushed stone).

Elimination or reduction of transportation expense for construction materials, particularly those which are used in great quantity, will effect a considerable reduction in the total cost of the project. The most economical type of dam will often be the one for which materials are to be found in sufficient quantity within a reasonable distance from the site.

The availability of suitable sand and gravel for concrete at a reasonable cost locally and perhaps even on property which is to be acquired for the project is a factor favorable to the use of a concrete structure. On the other hand, if suitable soils for an earthfill dam can be found in nearby borrow pits, an earthfill dam may prove to be the most economical. Advantage should be taken of every local resource to reduce the cost of the project without sacrificing the efficiency and quality of the final structure.

70. *Spillway Size and Location.* The spillway is a vital appurtenance of a dam. Frequently its size and type and the natural restrictions in its location will be the controlling factor in the choice of the type of dam. Spillway requirements are dictated primarily by the runoff and streamflow characteristics, independent of site conditions or type or size of the dam. The selection of specific spillway types will be influenced by the magnitudes of the floods to be bypassed. Thus, it can be seen that on streams with large flood potential, the spillway will become the dominant structure, and the selection of the type of dam could become a secondary consideration.

The cost of constructing a large spillway is frequently a considerable portion of the total cost of the development. In such cases, combining the spillway and dam into one structure may be desirable, indicating the adoption of a concrete overflow dam. In certain instances, where excavated material from separate spillway channels can be utilized in dam embankment, an earthfill dam may prove to be advantageous. Small spillway requirements often favor the selection of earthfill or rockfill dams, even in narrow damsites.

The advisability or practice of building overflow concrete spillways on earth or rock embankments has generally been discouraged because of the more conservative design assumptions and added care needed to forestall failures. Inherent problems associated with such designs are: Unequal settlements of the structure due to differential consolidations of the embankment and foundation after the reservoir loads are applied; the need for special provisions to prevent cracking of the concrete or opening of joints which could permit leakage from the channel into the fill, with consequent piping or washing away of the surrounding material; and the construction delays necessitated by the requirement for having a fully completed and seasoned dam before spillway construction can be started. Consideration of the above factors, coupled with increased costs brought about by more conservative construction details such as arbitrary increased lining thickness, increased reinforcement steel, cutoffs, joint treatment, drainage, and preloading, have generally led to selection of alternative solutions for the spillway design such as placing the structure over or through the natural material of the abutment or under the dam as a conduit.

One of the common spillway arrangements is the utilization of a channel excavated through one or

484556 O—60——7

both of the abutments outside the limits of the dam, or at some point removed from the dam. Where such a location is adopted, the dam can be of the nonoverflow type which extends the choice to include earthfill and rockfill structures. Conversely, failure to locate a site for a spillway away from the dam requires the selection of a type of dam which can include an overflow spillway. The spillway overflow can then be placed so as to occupy only a portion of the main river channel, in which case the remainder of the dam could be either of earth, rock, or concrete. (Olympus Dam (fig. 22) is an example.)

71. ***Earthquake.***—If the dam lies in an area that is subject to earthquake shocks, the design must include provisions for the added loading and increased stresses. The types of structures best suited to resist earthquake shocks without damage are earthfill and concrete gravity dams. For earthquake areas, neither the selection of type nor the design of the dam should be undertaken by anyone who is not experienced in this type of work.

C. LEGAL, ECONOMIC, AND ESTHETIC CONSIDERATIONS

72. ***Statutory Restrictions.***—Statutory restrictions exist with respect to control of the waters of navigable streams. Plans for diversion or control of waters in such streams are subject to approval by the Corps of Engineers, U.S. Department of the Army. There are numerous other Federal and State regulations relating to dam construction and operation which may be determining factors in the choice of type of structure. Almost every State has laws and regulations governing the design, construction, and operation of all dams and reservoirs of appreciable size. Engineers or owners considering dam construction in any of the 48 States will find concise information on what type of control is exercised by the State, as well as the name and address of the appropriate official and agency which exercises such control in the reference "Register of Dams in the United States." [2] The proper authorities should be consulted before proceeding with detailed designs.

73. ***Purpose and Benefit-Cost Relation.***—Consideration of the purpose which a dam is to serve will often suggest the type most suitable as, for example, whether its principal function is to furnish continuous and dependable storage of the water supply for irrigation, power, or domestic use; to control floods by detention; to regulate the flow of the streams; or to be a diversion dam or a weir without storage features.

Few sites exist where it would be impossible to build a dam that would be safe and serviceable, but in many instances conditions inherent in the site will result in a project cost in excess of the justifiable expenditure. The results of a search for desirable dam sites often determine whether a project can be built at a cost which will be consistent with the benefits to be derived from it. Accepted procedures are available for evaluating the benefits for waterpower, irrigation, and water supply uses; these procedures are less well defined for flood control; and there is no satisfactory measure of value for recreational projects.

Justification for recreational development must be evaluated on the basis of a comparison of the population that will be benefited, the location of other projects of the same kind, and the trend of development in the district (appreciative and depreciative)—all as related to the cost of the project and the money available. In a case where the need is great but the number of people to be served is limited, the development of an expensive site may not be justified. In another case, the present need may be great but a tendency toward decline of population and property values must be considered. In both instances, the development should be made as inexpensive as possible, probably with a low dam of small storage capacity.

74. ***Appearance.***—In general, every type of structure should have a finished, workmanlike appearance, compatible with its functional purpose. The alinement and texture of finished surfaces should be true to the design requirements and free from unsightly irregularities. Esthetic considerations may have an important bearing on the selection of type of structure, especially one which is designed primarily for recreational use.

[2] Mermel, T. W., "Register of Dams in the United States," McGraw-Hill Book Co., Inc., 1958.

Foundations and Construction Materials

DR. J. W. HILF [1]

A. SCOPE OF INVESTIGATIONS

75. *General.*— Information relating to foundation conditions and to the natural materials available for construction is essential for the design of small dams. The investigation is conducted in the office, in the field, and in the laboratory. For efficiency, the search for data must be properly planned. Subsurface explorations should not be started until all available geological and soils data have been evaluated. The investigator should know how to classify soils and rocks and should have an understanding of the geological and engineering characteristics of landforms. This background and a knowledge of the capabilities and limitations of the various methods of subsurface exploration will lead to selection of the most appropriate field methods, thereby avoiding time and effort lost through ineffectual procedures. The investigator should be familiar with logging and sampling methods and with the field and laboratory tests used for small dams.

The scope of investigation of foundations and of the various types of construction materials is given in this part of the chapter. Parts B through I provide information on the techniques and procedures for making these investigations.

76. *Foundations.*— Foundation investigations are necessary to ascertain whether a safe structure can be built at a selected site. The preferred type of structure can often be determined by inspection of the site, as described in chapter III. The construction of a dam whose failure would result in a destructive flood involves a serious public responsibility. A large number of damaging floods have been caused by failures of small dams. Investigations have shown that many of these failures were due to insecure foundations. A considerable number of failures attributed to other causes probably originated in defective foundations. It is undoubtedly true that many failures could have been averted by thorough investigations, which would have led to the selection of safe and satisfactory sites or the adoption of provisions in design and construction necessary to overcome defects in the foundation.

The first and one of the most important steps in the investigation of a reservoir is a reconnaissance for the purpose of selecting, principally upon the basis of topography and areal geology, the most favorable of the potential dam sites. Such a reconnaissance is a task for both the engineer and the engineering geologist and should be entrusted only to men experienced in work of this kind. The field work should be preceded by a study of all available data relating to the stream and to the area under consideration, including examination of maps, air photos, and reports, particularly those by the U.S. Geological Survey. Part B of this chapter discusses the various sources of information. A thorough reconnaissance leading to the selection of the most desirable site for the dam, or the elimination of as many of the potential dam sites as possible where there is a choice of sites, may save many thousands of dollars in exploratory work.

Foundation conditions are often revealed by or can be inferred from visual inspection of erosional features; rock outcrops; and manmade excavations such as highway or railroad cuts, building excavations, soil pits, and rock quarries. Some informa-

[1] Engineer, Earth Dams Section, Bureau of Reclamation

tion on ground-water conditions often can be obtained from local wells. The results of the field studies should be sketched preferably on a topographic map, although air photos can be used. At this stage of investigation, the sketch map often can show boundaries of soil deposits and rock outcrops, location of fault zones or other visible geologic irregularities, and the dip and strike of geologic features such as joints, bedding, and sheared zones.

The map should be accompanied by a report describing the various geologic features including rock and soil classifications, types of cementing materials that may occur in the rock and soil, and the origin and mode of deposition of the various soil deposits. The reconnaissance report should discuss the relationship of the geological conditions to the present and future permeability of the reservoir and dam foundation, and to the future stability and permanence of the dam, spillway, and other structures. Readily apparent geological problems requiring resolution by further investigations also should be discussed and a tentative program outlining the extent and character of more detailed explorations for the next stage of investigation should be recommended.

In the feasibility stage of investigation, subsurface exploration of the foundation is needed to determine definitely: (1) The depth to bedrock at the damsite, and (2) the character of both the rock and the soils under the dam and appurtenant structures. A line of holes usually is required at a damsite to determine the bedrock profile along the proposed axis. Since any axis selected in the field is necessarily tentative and subject to adjustment for design reasons, a few additional holes upstream and downstream from the axis are desirable. The number of holes required for foundation exploration of small dams is determined by the complexity of geological conditions, but the maximum spacing should not exceed 500 feet, and the depth of the holes should be at least equal to the height of the dam.

It is also necessary to develop the subsurface conditions at possible locations for the appurtenant structures, such as spillways, outlets, cutoff trenches, and tunnel portals, to clarify any unresolved geological problems. Holes in the areas of appurtenant structures usually should have a maximum spacing of 100 feet and should extend below the foundation at least 1½ times the base width of the structure. For the investigation of diversion dams, holes should be spaced a maximum of 100 feet apart in a pattern dictated by the complexity of the foundation. At least one hole should be located at each pier site.

Although there are many methods of penetrating overburden soils, only those that offer an opportunity for sampling and testing the foundation without excessive disturbance are recommended for exploring foundations for dams. Consequently, wash borings for example, are not discussed in this text. Test pits, trenches, tunnels, and large-diameter auger borings, which permit visual examination of the foundation, are excellent methods of determining the character of the overburden and are recommended wherever they are practicable. The recommended boring methods for exploring the foundation of small dams are rotary drilling and drive sampling (standard penetration test). Density in-place tests and determination of moisture content of soils above water table are desirable. Borings in rock require use of a rotary drill and diamond bits to obtain cores. Approximate values of the permeability of rock strata and soil overburden can be determined by water tests in bore holes. In all subsurface exploratory holes, it is important to measure and record the depths to water tables and the dates of these measurements.

The report prepared on completion of the feasibility stage should include a large-scale map showing surface geology, locations of all exploratory holes, and any geological sections measured in natural or manmade cuts. Cross sections should be prepared showing the known and probable subsurface geologic features. Logs of all holes should be included. Figure 23 is an example of a geologic map of a damsite, and a section along the centerline of the proposed dam.

For preparation of specifications, additional boreholes may be required in the foundations to clarify critical geological questions or to pinpoint depth to rock for preparation of detailed construction drawings. Additional samples and laboratory tests may be required to establish a firm basis for design of soil foundations.

77. *Embankment Soils.*—Some damsites require considerable excavation to reach a competent foundation and, in many cases, the excavated material is satisfactory for use in portions of the embankment. Excavations for spillway and out-

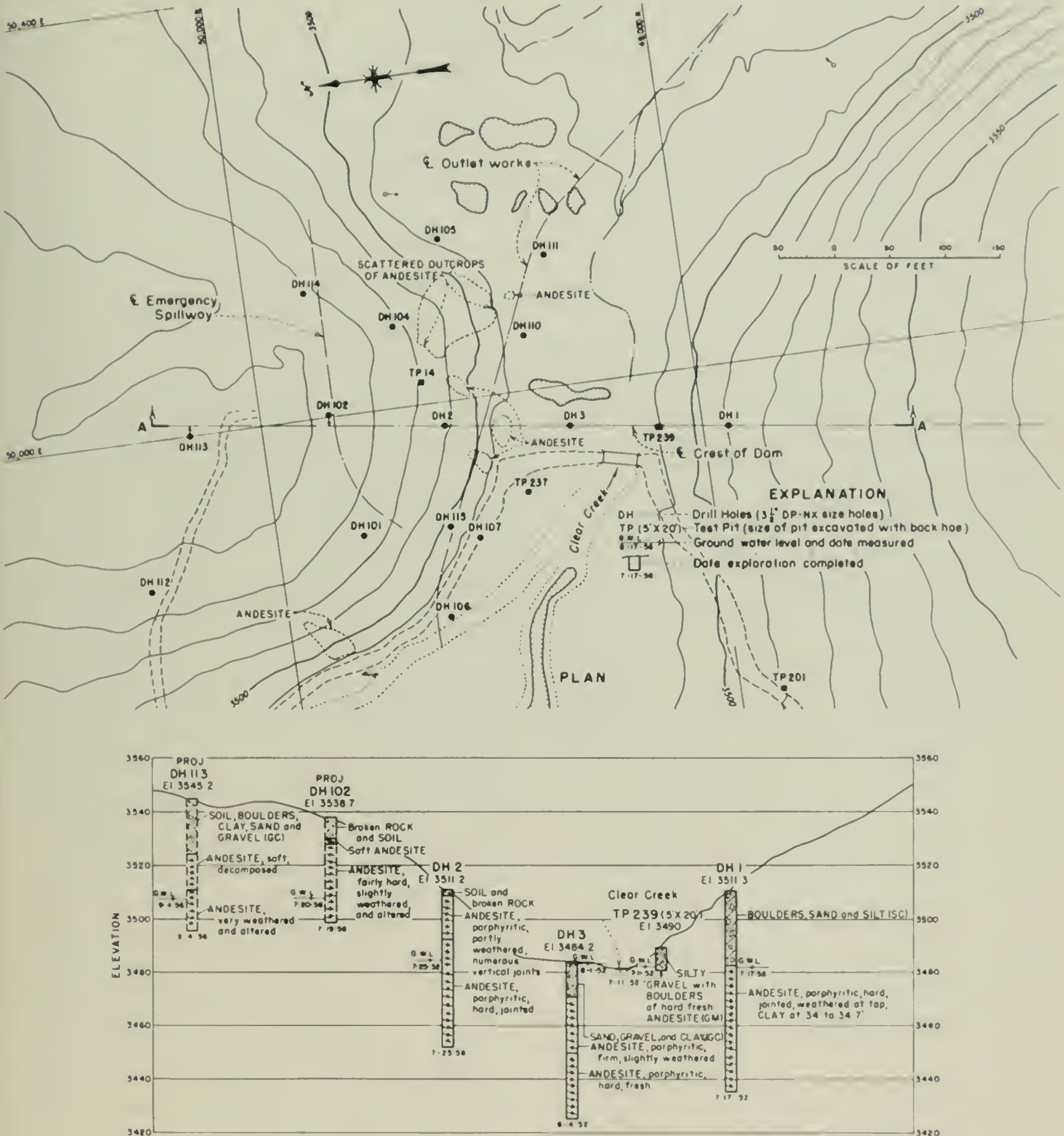


Figure 23. Geologic map and section of a damsite.

let works will also produce varying amounts of usable material. A major portion of embankment yardage for an earthfill dam, however, nearly always must be borrowed.

Investigation for embankment materials is a progressive procedure, ranging from a cursory inspection during the reconnaissance stage to extensive studies of all possible sources of material prior to undertaking the final design. A reconnaissance of the materials situation at each

prospective location should be made before a damsite is selected. Careful examination of existing maps, soil surveys, and air photos will usually locate the areas which should be examined in the field. Highway and railroad cuts, arroyos, and banks along stream channels will provide valuable clues to the nature of the materials underlying an area, and should be examined. It will rarely be necessary to excavate test pits or auger holes in the reconnaissance stage. Quantity determinations can be made by consideration of topographic features and by a few rough measurements, either on the ground or on maps. The reconnaissance report should include a sketch map showing the locations of available borrow areas with respect to the damsite, the character of the materials in each area, and the probable quantity of each. Local factors which would affect the use of a deposit should be discussed in the report. In addition to the engineering properties of the soils, many other facets should be considered, including: Proximity, accessibility, natural moisture content, and workability of the material; costs of rights-of-way and stripping; thickness of deposits; destruction of scenic features, and adverse topography.

A systematic plan for locating borrow areas should be followed during the feasibility stage of investigations subsequent to the selection of the damsite. To avoid overlooking nearby areas, the prospecting should start from the damsite and extend outward in all directions. All potential borrow areas within 1 mile of the dam should be investigated before more distant sources are considered. Holes should be dug at approximately 500-foot centers on a rough grid system in all practicable locations within the 1-mile limit. Earth augers should be used wherever possible, but test pits should be dug where boulders are encountered. Holes should extend 25 feet below ground surface except where bedrock or water table is encountered prior to reaching that depth. Holes should be sampled and logged in accordance with the procedures given in parts G and H of this chapter. Exploration at probable locations of cutoff trenches, for foundation stripping, and for spillways and outlet works should be given high priority in the investigation plan, and more detailed work in these areas is justified because of their possible early use as sources of embankment materials. When it becomes evident that a sufficient quantity of suitable material is not to be found within 1 mile of the damsite, more distant areas should be investigated according to the plan.

The ultimate purpose of a detailed borrow pit investigation is to determine the depth of shovel cut and the distribution of the materials in the embankment. This can be accomplished if a sufficient number of holes is dug to fix adequately the soil profiles in the borrow area. The plotting of profiles on at least 500-foot centers will indicate whether additional holes are needed. It is evident that the more homogeneous the soil is in a borrow area, the fewer holes will be required to establish the tentative profile. Figure 24 is an example of exploration for embankment materials for a dam. Soil classifications should be verified by laboratory tests on representative samples of the various materials encountered. A few density in-place tests should be made in each borrow area to determine the shrinkage factor to be applied between borrow pit and compacted embankment yardages. Procedures for this test are given in section 114.

Because of changes in plan, errors in estimating, and other contingencies, large safety factors should be used in estimating quantities available from borrow areas. The following criteria will assure adequate quantities: For a reconnaissance report, when it is estimated that less than 10,000 cubic yards of a type of material is needed, 10 times the estimated amount should be located; for requirements larger than 100,000 cubic yards, 5 times the estimated amount should be located. For feasibility estimates based on subsurface explorations, these safety factors can be cut in half. Even for well-explored borrow areas, at least 1½ times the required quantity customarily is outlined in the specifications in order to assure adequate quantities regardless of the contractor's choice of equipment and methods of excavation. Larger factors are often used when the existing information indicates the deposits are expected to be erratic.

78. *Riprap and Rockfill.*—Riprap is a layer of large, durable rock fragments. Its purpose is to preserve the shape of a surface slope or underlying structure by preventing erosion due to wave action or stream current. Rockfill is embankment constructed of rock fragments in portions of earthfill dams or in rockfill dams.

Search for suitable sources of riprap and rockfill is conducted in the same general sequence as is the search for earth embankment materials.

Since riprap is almost essential for an earthfill dam, it is impractical to limit the extent of the area to be searched for it. Prospecting should extend radially outward from the site of the proposed work until a deposit of rock is located which is suitable in quality and sufficient in volume to fulfill the anticipated requirements. The best possible use should be made of existing data, such as geologic maps, air photos, topographic maps, and publications of State, Federal, or private agencies. From a study of these data, existing quarries, outcrops, and other promising areas can be marked on the map or photograph for later investigation in the field. It is often profitable to inquire about rock deposits or accumulations from residents of the locality, and from officials of local governments.

The primary criteria for riprap are quality and size of the rock fragments. Those who perform the investigations should attempt, by inspection,

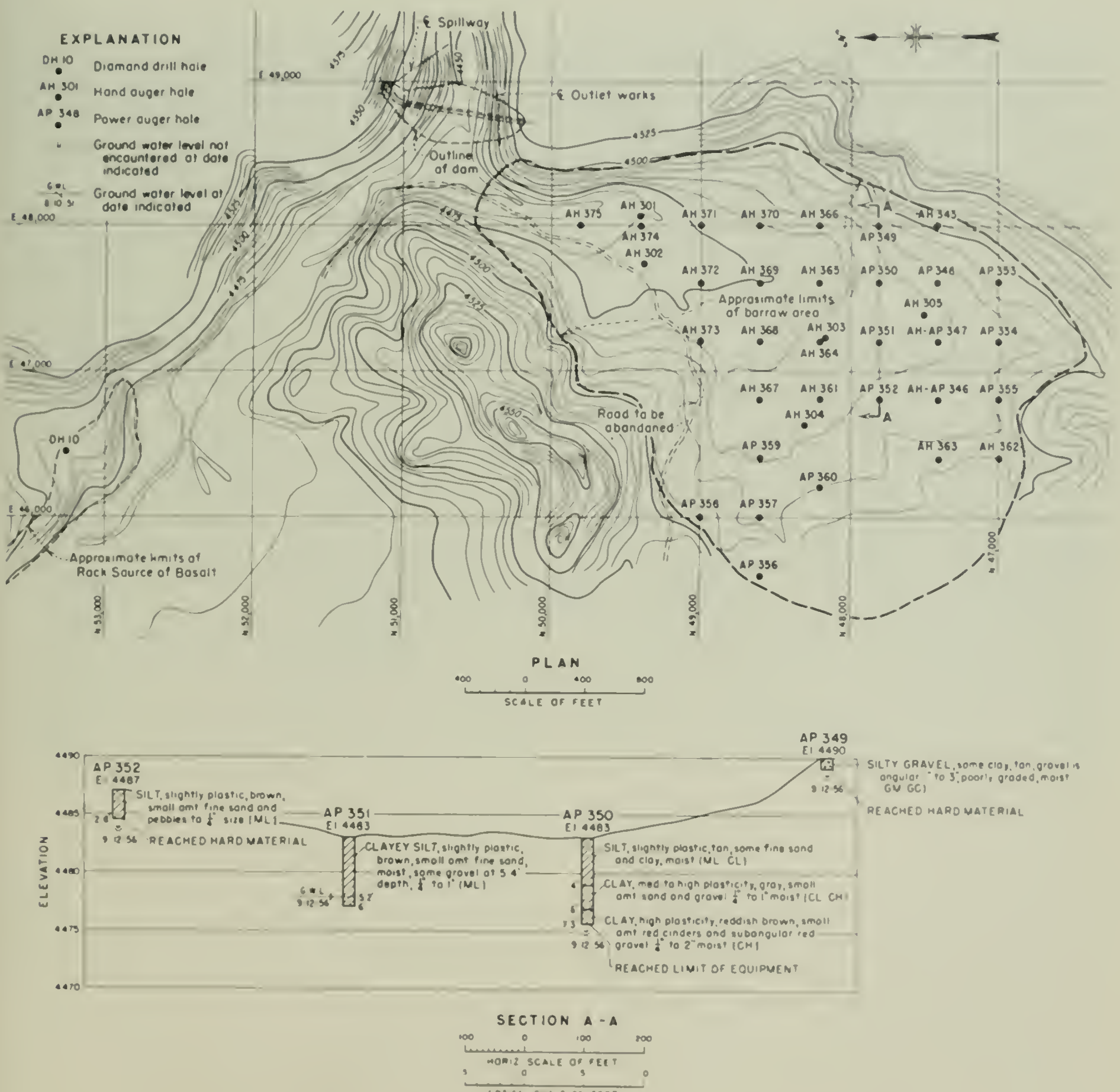


Figure 24. **Exploration for embankment materials—location map and section.**

to evaluate the ability of the rock to resist wave action, freezing and thawing, and other disintegrating forces, and to determine whether the deposit will yield sufficient material of the required sizes. The most obvious place to begin exploration of the rock source is at any outcrop. Existing vertical faces cut back to unweathered material should be thoroughly examined for strain cracks, fracture patterns, bedding planes, and zones of unsound material. The joint and bedding plane systems are especially important because they indicate the sizes which probably will be produced in quarrying. Exploration required for determining the characteristics of the unexposed portions of a potential riprap source is usually accomplished with boreholes, test pits, or trenches. Core drilling is normally the most practicable and reliable method of determining the extent of the deposit, the depth of overburden, and the thickness of rock. The cores recovered provide information about the quality of the rock in portions of the deposit not otherwise exposed. In the example shown in figure 24, only one borehole was needed in the rock source to establish the depth of the extensive exposed lava flow.

Where bedrock formations are not of suitable quality for riprap, other sources must be investigated. There have been several cases where surface boulders were gathered and used for riprapping earthfill dams because quarry rock of suitable quality could not be found even within 100 miles of the site. The use of this type of riprap is normally feasible only when the boulders occur in fairly well concentrated accumulations, and when they cover areas sufficiently large to provide significant quantities. Nevertheless, the exploitation of several widely separated accumulations to yield the quantity required for one dam is not uncommon.

Occasionally, talus slopes are found which contain durable rock of the required sizes, and which are of sufficient extent to make quarrying from the source deposit unnecessary. Such slopes are especially desirable when they are more accessible than the source deposit, as is often the case. Little can be done in the way of exploration of talus material except for making a thorough examination and survey to determine the characteristics of the rock, the quantity available, and the range of sizes. Good photographs are part of the exploratory data for all riprap and rock sources, and they are especially valuable when talus slopes are being considered. Figure 25 shows a talus deposit of igneous rock suitable for riprap.

The availability of riprap or rockfill materials has a pronounced effect on the design of a structure; consequently, very careful studies of quantities must be made. It is occasionally possible to make use of a readily accessible material rather than to require the procurement of a superior rock at considerably greater expense. On the other hand, the use of lesser quantities of superior materials might offset their added cost. Information on the sampling of riprap sources is given in part G of this chapter. The suitability of rockfill materials is based on identification and field examination of the source.

79. *Concrete Aggregate.*—Field investigations for concrete materials prior to construction are confined chiefly to prospecting for aggregate and to explorations and sampling of available deposits. Those engaged in prospecting work should be familiar with the effects of grading, physical characteristics, and composition of aggregate on the properties of concrete. Judgment and thoroughness in conducting preliminary field investigations are usually reflected in durability and economy of the finished structures.

Most factors pertaining to the suitability of aggregate deposits are related to the geological history of the region. These factors include size, shape, and location of the deposit; thickness and character of the overburden; types and condition of the rock; grading, rounding, and degree of uniformity of the aggregate particles; and groundwater level. Aggregate may be obtained from deposits of natural sand and gravel, from talus, or from quarries in areas of bedrock outcroppings. Fine blending sand may sometimes be obtained from windblown deposits.

Stream deposits are the most common and generally most desirable because: (1) Individual pieces are usually rounded; (2) streams exercise a sorting action which may improve grading; and (3) abrasion caused by stream transportation and deposition leads to a partial elimination of weaker materials. Alluvial fans are frequently used as sources of aggregate, but they often require more than usual processing. Glacial deposits provide sand and gravel, but they are generally restricted

to northerly latitudes or high elevations. Those glacial deposits not influenced by fluvial agencies are usually too heterogeneous to be suitable as aggregate and at best are usable only after elaborate processing. Glacial deposits which have been affected by stream action frequently yield satisfactory aggregate materials.

When natural sand and gravel are not available it is necessary to produce concrete aggregate by quarrying and processing rock. Quarrying normally is done only when other materials of adequate quality and size cannot be obtained economically. Quarry deposits frequently contain stratified materials which make it difficult to obtain representative samples of the undeveloped source. Also, the presence of layers or zones of undesirable materials, such as clay or shale, in some instances necessitates selective quarrying and special processing.

The extent and justifiable expense of exploration for concrete aggregate are determined largely by the size of the job and the purpose of the structure. When searching for suitable aggregate it is important to remember that ideal materials are seldom found. Deficiencies or excesses of one or more sizes are very common. Objectionable rock types, coated and cemented particles, or particles of flat or slabby shape may occur in excessive amounts.

Figure 25. **Talus slope of igneous rock suitable for riprap.**

The most promising deposits should be explored and sampled by means of cased test holes, open test pits, or trenches, and the suitability of the aggregate should be determined. The methods of subsurface exploration, sampling, logging, and testing are given in parts F, G, H, and I, respectively, of this chapter. The quality and gradation requirements for aggregates are discussed in appendix F.

B. SOURCES OF INFORMATION

80. ***Topographic Maps.***—A topographic map is indispensable in the design and construction of a dam. Before undertaking the job of map making, a thorough search should be made for the existence of maps covering the areas of the reservoir, the damsite, and potential sources of construction materials. The U.S. Geological Survey (USGS) should be contacted for information on the availability of maps. This organization is making a series of standard topographic maps to cover the United States, Alaska, Hawaii, and Puerto Rico.

The unit of survey for the USGS maps is a quadrangle bounded by parallels of latitude and meridians of longitude. Quadrangles covering 7.5 minutes of latitude and longitude are generally published at the scale of either 1:24,000 (1 inch equals 2,000 feet) or 1:31,680 (1 inch equals ½ mile). Quadrangles covering 15 minutes of latitude and longitude are published at the scale of 1:62,500 (1 inch equals approximately 1 mile), and quadrangles covering 30 minutes of latitude and longitude are published at the scale of 1:125,000 (1 inch equals approximately 2 miles). In certain Western States, a few quadrangles covering 1° of latitude and longitude have been published at the scale of 1:250,000 (1 inch equals approximately 4 miles). A few special maps are published at other scales. Each quadrangle is designated by the name of a city, town, or prominent national feature within it, and on the margins of the map are printed the names of adjoining quadrangle maps that have been published.

The distinctive characteristic of topographic maps is that the shape of the land is portrayed by contours, which are imaginary lines following the ground surface at a constant elevation. The contour interval is the regular elevation difference separating adjacent contour lines on the map. These intervals depend on the ground slope and the map scale; they vary from 5 feet to 200 feet. On some quadrangle maps, devices other than contours are used to show topographic forms. These devices are hachures, form lines, symbol patterns, and relief shading.

Published maps of the U.S. Geological Survey are colored to distinguish classes of map features. Black is used for manmade or cultural features, such as roads, dams, buildings, names, and boundaries. Blue is used for water or hydrographic features, such as lakes, rivers, canals, and glaciers. Brown is used for relief or hypsographic features—land shape portrayed by contours or hachures. Green is used for wooded areas with typical pattern to show scrub, vineyards, and orchards. Red emphasizes important roads, and shows built-up urban areas and public land subdivision lines.

In addition to the published topographic map, information of great assistance to engineers is available from the U.S. Geological Survey for mapped areas. For example, the locations and true geodetic positions of triangulation stations and permanent benchmarks are recorded. Also, map manuscripts four times larger in scale than published maps may be available 1 or 1½ years prior to publication of the final map. Figure 26 shows the status of topographic mapping in the United States, mainly of the 7½- and 15-minute series, distributed by the U.S. Geological Survey. A large index map similar to this is available without charge.

In the absence of topographic coverage of the area, other types of maps may be used in the preliminary stages. Of considerable importance to dam design are the river survey maps. These are strip maps which show the course and fall of the stream; the configuration of the valley floor and the adjacent slopes; and the locations of towns, scattered houses, irrigation ditches, roads, and other cultural features. River survey maps were prepared largely in connection with the classification of public lands, hence most of them show areas in the Western States. If the valley is less than a mile wide, the topography is shown to 100 feet or more above the water surface; if the valley is flat and wide, topography is shown for a strip of 1 to 2 miles. Potential reservoir sites are mapped to the probable flow line of the reservoir. The usual scale is 1:31,680 or 1:24,000, and the normal contour interval is 20 feet on land and 5 feet on the water surface. Many of these maps include proposed dam sites on a large scale and have a profile of the stream. The standard size sheet is 22 by 28 inches.

The availability of river survey maps, other special maps and sheets, including national parks and monuments, and a list of agents for topographic maps are indicated on the index to topo-

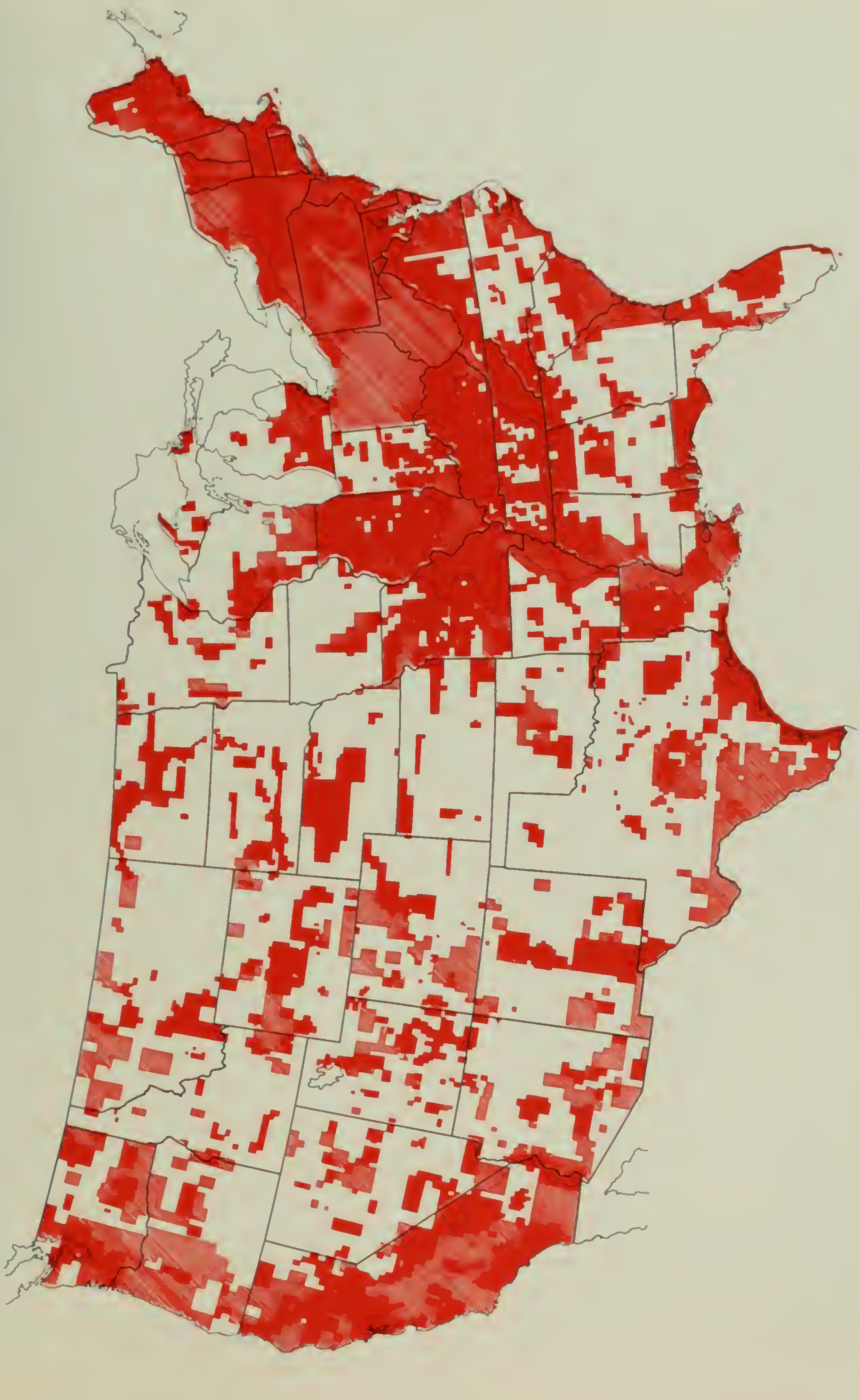


Figure 26. Status of topographic mapping in the United States, 1956. Solid color indicates map coverage of standard quality. Crosshatching indicates coverage of other quality. (Geological Survey.)

graphic mapping for the various States. These indices are available without charge from the U.S. Geological Survey. Requests and inquiries on published maps and on the availability of map manuscripts or other information of the area should be directed to U.S. Geological Survey, Denver Federal Center, Denver, Colo., or Washington 25, D.C.

Topographic maps are of considerable value in the exploration of foundations and construction materials for dams. The locations and elevations of exploratory holes, outcrops, and erosional features can be placed on the map, and the landforms portrayed by the contours indicate to some degree the type of soil. Information on the origin and characteristics of some of the simpler landforms is given in part E of this chapter.

81. ***Geologic Maps.***—Considerable useful engineering information is obtainable from geologic maps. These maps identify the rock units directly underlying the reservoir and the dam site. The characteristics of rocks are of major importance in the selection of a dam site and in the design of the dam. Many surface soils are closely related to the type of rock from which they are derived. When the influence of climate and relief are considered, the engineer can make reasonable predictions of the type of soil associated with different parent materials. Conditions beneath the surface can often be correctly deduced by the 3-dimensional information given on geologic maps. These maps are especially valuable in areas where only limited information on soils from the agricultural standpoint is available; for example, in arid or semiarid regions where soils are thin.

On geologic maps rocks are identified by their geologic age. The smallest rock unit mapped is generally a formation. This is an individual bed or several beds of rock that extend over a fairly large area and that can be clearly differentiated from overlying or underlying beds because of a distinct difference in lithology, structure, or age. The areal extent of these formations is indicated on geologic maps by means of letter symbols, color, and symbolic patterns.

Letter symbols indicate the formation and geologic period. For example, "Jm" stands for the Morrison formation of the Jurassic period. Standard color and pattern conventions are followed on maps produced by the U.S. Geological Survey. Tints of yellow and orange are used for different Cenozoic rocks; tints of green for Mesozoic rocks; tints of blue and purple for Paleozoic rocks; and tints of russet and red for pre-Cambrian rocks. The primary structural features of the rock types are depicted, as far as practicable, by conventional patterns. Variations of dot and line patterns are used for sedimentary rocks; wavy lines for metamorphic rocks; and checks, crosses, or crystallike patterns for igneous rocks. Another type of symbol common on geologic maps indicates the attitude of the structural features of the rock strata. Structural symbols are given as marginal data for the geologic map. One of the most important symbols is the dip-strike symbol, which indicates the direction of strike of a rock bed, fault, fold, or flow structure; the direction of dip; and the angle of dip from the horizontal in degrees.

Geologic maps often carry one or more geologic sections as marginal data. The section is a graphic representation of the disposition of the various strata in depth along an arbitrary line usually marked on the map. Geologic sections are somewhat hypothetical, and must be used with caution. The vertical scale is nearly always exaggerated. Sections prepared solely from surface data may easily be erroneous; sections prepared from boring records or mining evidence are more reliable. A section compiled from information obtained in one small locality is called a columnar section, and shows only the succession of strata and not the structure of the beds as does the geologic section.

There are several types of geologic maps. A map showing a plan view of the bedrock in the area is a *bedrock* or *areal geologic map*. Such a map indicates the boundaries of the visible formations, and the inferred distribution of those units covered by soil or plant growth, and it usually includes one or more geologic sections. Except for indicating thick deposits of alluvium, areal maps do not show soil or unconsolidated mantle. In areas of complex geology where exposures of bedrock are scarce, the location of the contacts between formations is often indicated as approximate or hypothetical. *Surficial geologic maps* differentiate the unconsolidated surface materials of the area according to their geologic categories, such as stream alluvium, glacial gravel, and windblown sand. These maps indicate the areal extent, characteristics, and geologic age of the surface materials. Areal (bedrock) geologic maps of

moderately deformed areas often carry enough structural symbols to provide an understanding of the structural geology of that region. In highly complex areas, however, where a great amount of structural data is necessary for an interpretation of the geology, special *structural geologic maps* are prepared.

In addition to giving the geologic age of the mapped rocks, some maps briefly describe the rocks. Many maps, however, lack a lithologic description. The experienced geologist can make certain assumptions or generalizations from the age of the rock alone by making analogies with other areas. For more certain identification of the lithology and for details, geologic literature on the whole area must be consulted. Engineering information can be obtained from geologic maps if the user possesses a knowledge of the fundamentals of geology and an understanding of how engineers use geologic facts in design and construction. By a study of the basic geologic map, together with all the collateral geologic data that pertain to the area shown, it is possible to prepare a special map that interprets the geology in terms of construction materials. Similarly, foundation and excavation conditions, as well as surface- and ground-water data, can be interpreted from geologic maps. Such information is valuable in preliminary planning activities, but is not a substitute for detailed field investigations in the feasibility and specifications stages.

The U.S. Geological Survey now publishes a series of geologic quadrangle maps which replaces the earlier folios of the Geologic Atlas of the United States, published from 1894 to 1945. The new series consists of geologic maps supplemented where possible by structural sections and other graphic means of presenting geologic data, and accompanied by a brief explanatory text to make the maps useful for general scientific, economic, and engineering purposes. Full descriptions of the areas shown on these maps and detailed interpretations of geologic history are reserved for other publications, such as the bulletins and professional papers of the Geological Survey.[2] Separate maps of some quadrangles are published in the geologic quadrangle map series under such titles as "Economic Geology," "Surficial Geology," and "Engineering Geology." Each map is issued in two forms: Flat for filing in large map cases, and folded for use in the field.

There are several geologic maps of special interest to designers of dams. There is a series of maps resulting from geologic mapping and general resources investigations conducted by the Geological Survey as part of the Department of the Interior plan for study and development of the Missouri River Basin. These include maps showing construction materials and nonmetallic mineral resources, including sand and gravel deposits of several of the States in the Missouri River Basin. The Geological Survey has also published a bound set of six maps entitled "Interpreting Geologic Maps for Engineering Purposes, Holidaysburg Quadrangle, Pennsylvania," 1953, containing examples of how geologic maps are used to solve engineering problems including a problem of selection of a damsite.

Figure 27 shows the status of geologic mapping in the United States. More detailed information about published geologic maps for individual States is given in the series of geologic map indexes available from the U.S. Geological Survey. Each published geologic map is outlined on a State base map, with an explanatory key giving the source and date of publication, the author, and the scale. The attention of all those engaged in searching for geological information should be called to the Directory of Geological Material in North America [1].[3] This directory includes comprehensive lists of sources of available maps, charts, air photos, logs, cores, etc., for each State and Territory of the United States, as well as for provinces in other countries of North America.

82. *Agricultural Soil Maps.* —A large portion of the United States has been surveyed by the Department of Agriculture. These investigations are surficial, extending to depths up to 6 feet, and consist of classifying soils according to color, structure, texture, physical constitution, chemical composition, biological characteristics, and morphology. The Department of Agriculture publishes reports of their surveys in which the different soils are described in detail, and their suitability for various crops is given. Included in each report is a map of the area surveyed, usually a county, showing by the pedological classification the various types of soils that occur. In addition to the county soil maps, there are many areas in which

[2] Publications of the U. S. Geological Survey are available from the U. S. Government Printing Office, Washington 25, D. C.

[3] Numbers in brackets represent items in the bibliography, sec. 117

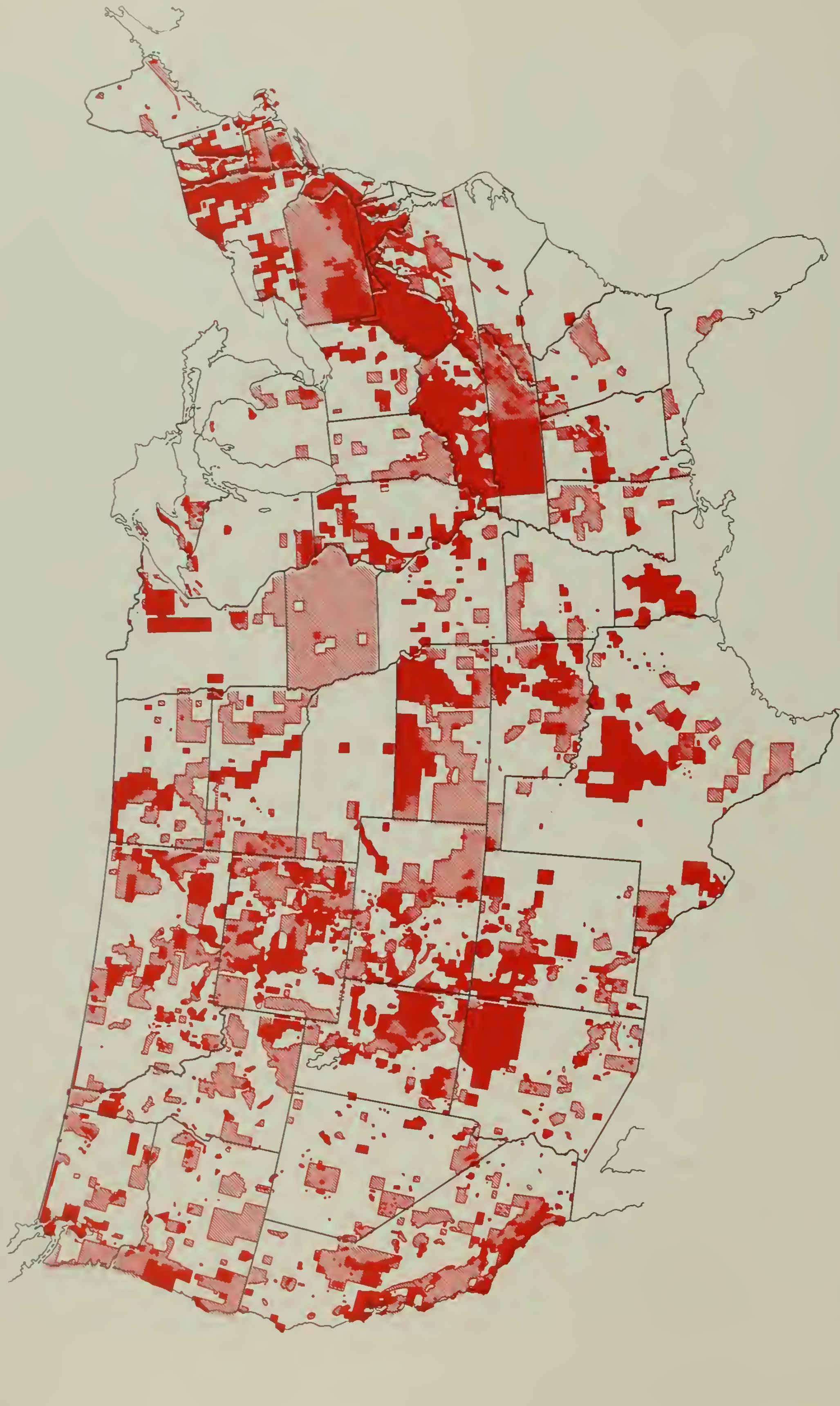


Figure 27. Status of geologic mapping in the United States, 1956. Solid color indicates areas mapped at scales of 1 mile to 1 inch or larger. Crosshatching indicates areas mapped at scales smaller than 1 mile to 1 inch, up to and including 2 miles to 1 inch. (Geological Survey.)

individual farms are mapped using the same system of soil classification.

Figure 28 shows the extent of published agricultural soil mapping in the United States. These surveys (if in print) are available for purchase from the Superintendent of Documents, Washington, D.C. Out-of-print maps and other unpublished surveys may be available for examination from the U.S. Department of Agriculture, county extension agents, colleges, universities, and libraries. Soil surveys using the agricultural soil classification have been made in many of the river basins in the 17 Western States for the purpose of classifying land for irrigation based on physical and chemical criteria. Inquiry should be made at the local project offices of the Bureau of Reclamation for the availability of soils data for these areas.

In order to apply agricultural soil maps to explorations of foundations and construction materials, some knowledge of the pedological system of classification is necessary. This system recognizes the fact that movement of water from the surface of the soil downward leaches inorganic colloids and soluble material from the upper portion to create a soil profile. The depth of leaching action depends on the amount of water, on the permeability of the soil, and on the length of time involved. This action produces distinct layers of soil. The surface layer is lacking in the fines which the subsurface layer has accumulated in addition to its original fines. The soil beneath the subsurface layer has been little affected by water and remains essentially unchanged. These three layers are designated from the surface downward as the A horizon, the B horizon, and the C horizon. In detailed classifications these horizons may be subdivided into A_1, A_2, etc.

The soils of the United States are first divided into main divisions depending on the cause of profile development and on its magnitude. The main soil divisions are further divided into suborders and then into great soil groups on the basis of the combined effect of climate, vegetation, and topography. Within each great soil group the soils are divided into *soil series*, each of which has the same age, climate, vegetation, relief, and parent material. According to this system of classification, all soil profiles of a certain soil series are similar in all respects, with the exception of a variation in the texture, or grain size, of the topsoil or A horizon. The soil series were originally named after a town, county, stream, or similar geographical source where the soil series was first identified.

The final classification unit, which is called the soil type, is made up of the soil series name plus the textural classification of the topsoil or A horizon. This textural classification is different from the Unified Soil Classification System used for engineering purposes (part C of this chapter). Figure 29 shows the textural classification of soils of the U.S. Department of Agriculture [2]. The chart shows the terminology used for different percentages of clay (defined as particles smaller than 0.002 mm.), silt (0.002 to 0.05 mm.), and sand (0.05 to 2.0 mm.). Note the use of the term "loam" which is defined in the chart as a mixture of sand, silt, and clay within certain percentage limits. Other terms used as adjectives to the names obtained in the triangle classification are: "Gravelly" for rounded and subrounded particles from 2 millimeters to 3 inches, "cherty" for gravel sizes of chert, and "stony" for sizes greater than 10 inches.

The textural classification given as part of the soil name on the agricultural soil map refers to the material in the A horizon only; hence, this is not of much value to the engineer who is interested in the entire soil profile. The combination of soil series name and textural classification to form a *soil type*, however, provides a considerable amount of significant data. For each soil series the texture, degree of compaction, presence or absence of hardpan or rock, lithology of the parent material, and chemical composition can be obtained. From the engineering point of view, this information is qualitative rather than quantitative, but it can often be used to advantage in the reconnaissance stage and in planning subsurface explorations for dams.

The use and limitation of agricultural soil classifications for engineering purposes can be judged by the following example taken from the Soil Survey Manual [3]. The "Cecil series" is described by a paragraph giving the great soil group to which it belongs, the general geographical distribution of the series, the rocks from which it was derived, and a comparison of the series with associated or related soil series. Additional paragraphs discuss the range in characteristics of the principal soil types of the Cecil series, and also relief, drainage, vegetation, land use, distribution

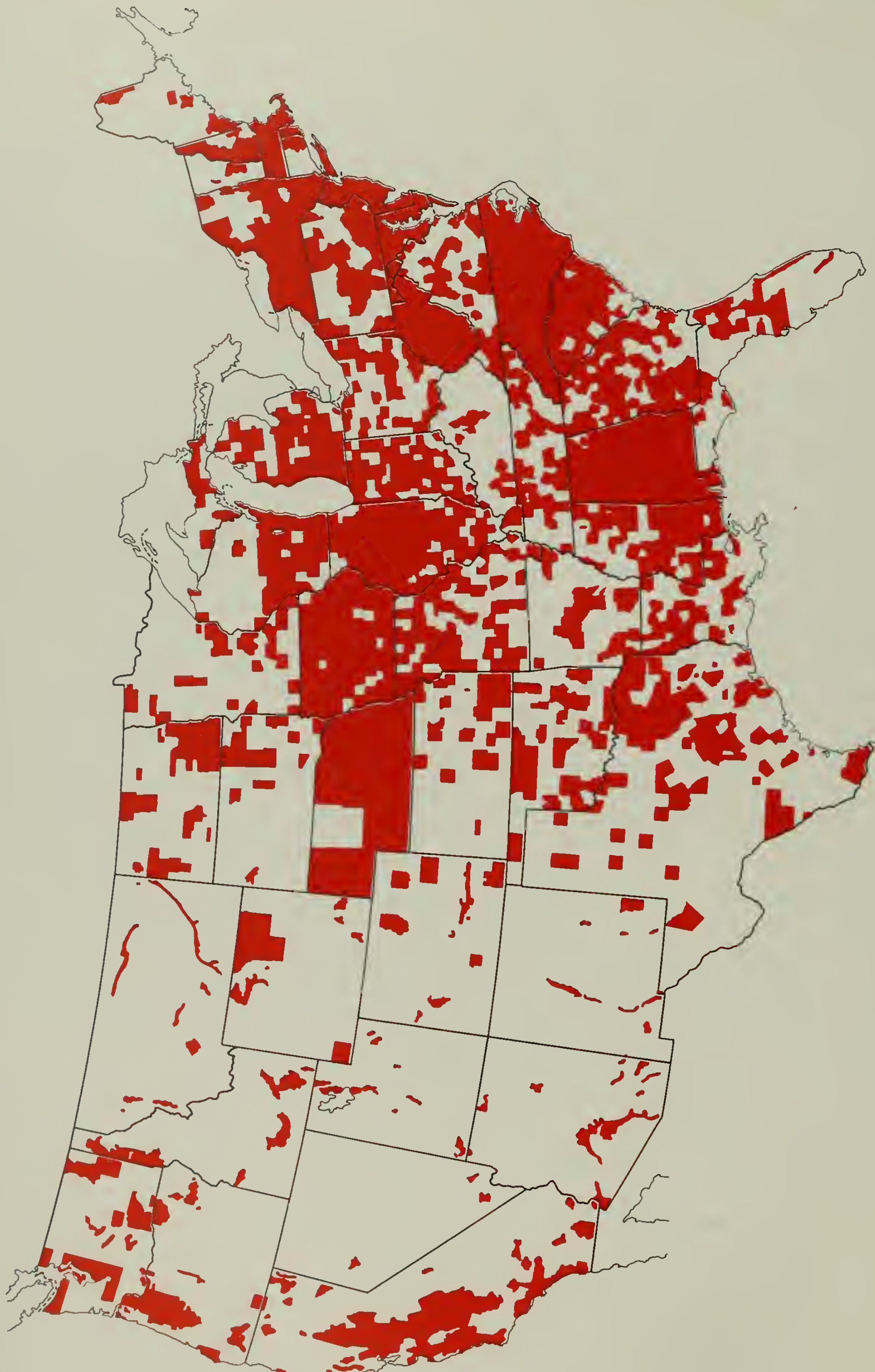


Figure 28. Status of published agricultural soil maps, 1956. (U.S. Soil Conservation Service.)

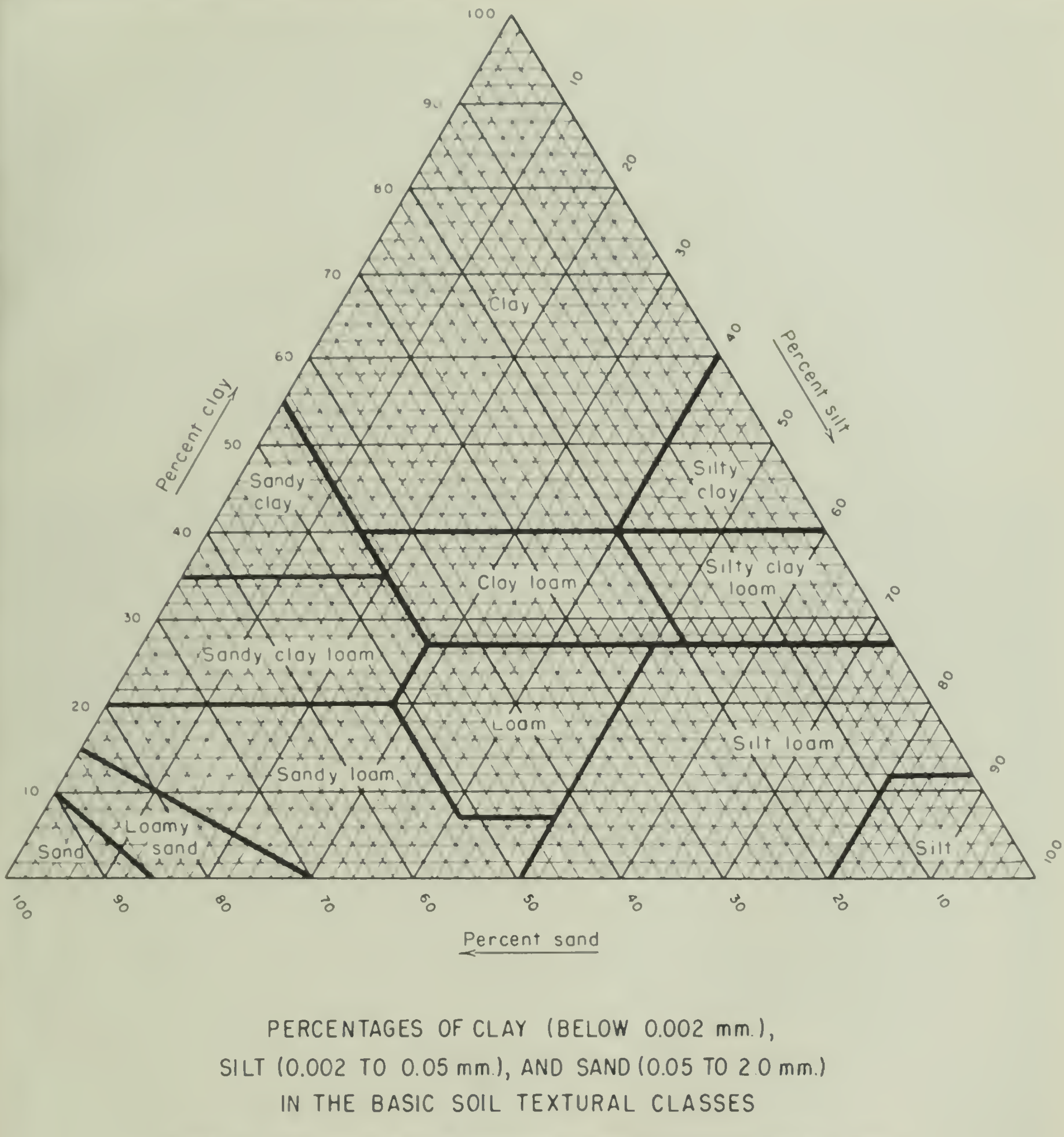


Figure 29. Soil triangle of the basic soil textural classes. (U.S. Soil Conservation Service.)

of the series by States, type location, and remarks. A soil profile for the Cecil series is given as follows:

Soil profile (Cecil sandy loam):

A_{00} A thin layer of leaves and pine needles.

A_1 0–2 inches, brownish-gray, very friable sandy loam with fine, weak, crumb structure; strongly acid. 1 to 4 inches thick.

A_2 2–8 inches, weak-yellow to light yellowish-brown, nearly loose or very friable sandy loam; strongly acid. 4 to 10 inches thick.

B_1 8–10 inches, weak reddish-brown to strong-brown, friable heavy sandy loam or light sandy clay loam with medium granular structure; strongly acid. 2 to 4 inches thick.

484550 O—60——8

B_2 10–38 inches, moderate to strong reddish-brown clay that is plastic when wet, very firm when moist, and very hard when dry. The clay has a medium moderately blocky structure and contains some white sand grains and small mica flakes; strongly acid. 20 to 36 inches thick.

B_3 38–60 inches, light to moderately reddish-brown clay loam with mottles or splotches of yellow; firm to friable when moist. The soil contains enough small mica flakes to make it feel slick when rubbed between the fingers; it has a weak, coarse, blocky structure and is strongly acid. 10 to 30 inches thick.

C 60 inches plus, mottled or splotched light reddish-brown, yellowish-brown, light-gray, and black friable disintegrated rock material in which there is usually much mica; strongly acid. 20 to 60 inches thick.

The agricultural soil survey report is designed to include information useful to the farmer and to the agricultural community. However, apart from the soil maps and soil profile descriptions contained in these reports, other information of great value in planning of reservoirs is included. The reports discuss topography; ground surface conditions; obstructions to movement on the ground; natural vegetation; size of farms; land utilization; farm practice and cropping systems; meteorological data; drainage; flood danger; irrigation; water supply and quality; nearness to towns, roads, and railroads; electric power; and similar data.

83. *Airphotos.*—An airphoto or aerial photograph is a pictorial representation of a portion of the earth's surface taken from the air. It may be a vertical photograph in which the axis of the camera is vertical, or nearly so, or an oblique photograph where the axis of the camera is more or less inclined. High oblique photographs include the horizon; low obliques do not. The vertical photograph is commonly used as the basis for topographic mapping, agricultural soil mapping, and geological interpretations.

Except where dense forest cover obscures large areas from view, the airphoto reveals every natural and manmade detail on the surface of the ground. Relationships are exposed which could not be found on the ground no matter how careful the examination. Identification of features shown on the photo is facilitated by stereoscopic examination. The features are then interpreted for a particular purpose, such as geology, land utilization, or engineering characteristics. The experience and training of the engineer will determine his utilization of aerial photographs. Knowledge of the elements of geology and of soil science will assist him in interpreting airphotos for engineering uses. Airphotos are very often used for locating areas to be examined and sampled in the field and as substitutes for maps.

Virtually the entire area of the United States has been covered by aerial photography. A 28- by 42-inch index map of the United States is available free on application to the U.S. Geological Survey, Washington 25, D.C. This map shows which of seven Government agencies can provide prints for particular areas. When ordering photographs, specify contact prints or enlargements, glossy or matte finish, and location. Location should be given by range, township, and section, latitude and longitude, State and county, or shown on an enclosed index map of the area. Stereoscopic coverage should be requested for most uses. Aerial mosaics covering roughly 25 percent of the area of the United States are also available. A mosaic is an assemblage of aerial photographs whose edges have been torn or cut, and matched and mounted to form a continuous representation of the earth's surface. They include halftone photolithographic reproductions from mosaic negatives known as "photo maps." An index map showing the status of aerial mosaics of the United States, including the coverage and the agencies holding mosaic negatives, is available without charge on application to the U.S. Geological Survey.

Airphoto interpretation of earth materials and geologic features is relatively simple and straightforward, but requires experience. The diagnostic features include terrain position, topography, drainage and erosional features, color tones, and vegetative cover. Interpretation is limited mainly to surface and near-surface conditions. There are special cases, however, where features on the photograph permit reliable predictions to be made of deep, underground conditions. Although interpretation can be rendered from any sharp photograph, the scale is a limiting factor, since small-scale photos limit the amount of detailed information that can be obtained. The scale of

1 : 20,000 has been found satisfactory for engineering and geologic interpretation of surface materials. Large-scale photos often have application to highly detailed work, such as for reservoir clearing estimates, and for geologic reconnaissance mapping of dam sites.

Airphotos can be used to identify certain terrain types and land forms. These topographic features are described in part E of this chapter. Stereoscopic photo inspection of an area, taking particular note of regional topography, local terrain features, and drainage conditions, will usually suffice to identify the common terrain types. This permits the possible range in the soil and rock materials to be anticipated, and their characteristics to be defined within broad limits.

Geologic features that may be highly significant to the location or performance of engineering structures sometimes can be identified from airphotos. In many instances these features can be more readily identified on the airphoto than on the ground. It must be recognized, however, that airphoto interpretation is applicable only to those features which develop recognizable surface expressions, such as drainage patterns and alinement of ridges or valleys. Joint systems, landslides, fault zones, folds, and other structural features sometimes are identified quickly in an aerial photo, whereas it may be difficult to find them on the ground. The importance of these items in the location of a dam and its appurtenant works is obvious. The general attitude, bedding, and jointing of exposed rock strata, as well as the presence of dikes and intrusions, often can be interpreted in airphotos. Such

Figure 30. Rock strata illustrating folding in sedimentary rocks. (A) Satanka formation, (B) Lyons formation, (C) Morrison formation, and (D) Lower and Middle Dakota formation. (U.S. Forest Service.)

Figure 31. **Sinkhole plain indicating deep plastic soils over cavernous limestone, developed in humid climate. (U.S. Commodity Stabilization Service.)**

information is valuable in appraising the possibilities of landslides into open cuts and of seepage losses in reservoirs.

Drainage patterns, particularly their type and density, provide an indication of the relative permeability of the earth materials. A dense, finely divided drainage pattern indicates an impervious soil area with high runoff and low infiltration. In contrast, the absence of a surface drainage pattern indicates a soil area with low runoff and high infiltration, provided the area is not a desert. The surface drainage pattern in areas of high water table has only limited significance as an indicator of the earth materials present. Definite alinements in the drainage pattern usually indicate control by local geologic structure.

Erosional features have significance in that they often reflect the textural characteristics of the exposed materials. Short, steep, V-shaped gullies with uniform gradients are associated with granular materials; long gullies with uniform gradients of rounded cross-sectional slopes are associated with fine-grained plastic soils. Silts and sand-clay materials usually exhibit gullies having U-shaped cross sections and compound gradients. The significance of gullies as an indicator of soil texture is modified by extreme climatic influence, such as in arid regions where "box" gullies seem to prevail irrespective of soil texture. Regardless of the climatic influence, however, changes in the gradient or cross section of gullies, or changes in the surface slope of eroded surfaces may indicate a change in the exposed soil, rock texture, or geologic structure.

Color tones (relative photographic gray values) have a general significance in that they reflect the

soil moisture conditions and often reveal the relative position of the ground-water table. Light color tones are usually associated with well-drained soils, such as gravels, sands, and silts with ground-water levels well below the ground surface. Dark color tones usually indicate poorly drained organic clays and silty clays with ground-water levels near the ground surface. The significance of soil color in airphotos must be appraised from the overall color pattern, since some variation may be expected in the photographic tone quality of individual airphotos. It is also necessary to exclude, visually, the color tones produced by vegetative cover.

Vegetative cover is significant in that the vegetative patterns produced in the airphotos often reflect the nature of soil and moisture conditions. Also, a change in vegetative pattern may indicate a change in the type or texture of the underlying bedrock. The use of vegetative patterns as an indicator of soil conditions will prove most useful in extreme climates, such as in arctic, tropical, and arid regions where the combination of soil and climate is selective of vegetative growth. In arid regions the pattern of vegetation can be used to distinguish between high and low alkali soils and between high and low ground-water levels. The effective use of vegetation as an airphoto materials indicator requires a limited amount of field correlation.

Figures 30 and 31 are examples of readily identifiable geologic features from airphotos. Examples of typical land forms on airphotos are given in part E of this chapter.

C. SOIL CLASSIFICATION

84. *General.*—Most soils are a heterogeneous accumulation of mineral grains that are not cemented together. However, the term "soil" or "earth" as used by engineers includes virtually every type of uncemented or partially cemented inorganic and organic material found in the ground. Only hard rock which remains firm after exposure is wholly excluded. To the engineer engaged in design and construction of foundations and earthworks for dams, the physical properties of soils, such as unit weight, permeability, shearing strength, compressibility, and interaction with water, are of primary importance.

It is advantageous to have a standard method of identifying soils and classifying them into categories or groups which have distinct engineering properties. This enables engineers in the design office and those engaged in field work to speak the same language, thus facilitating exchange of information and experiences. Knowledge of soil classification, including typical engineering properties of soil of the various groups, is especially valuable to the engineer engaged in prospecting for earth materials or investigating foundations for structures. To a limited extent proper soil classification can be used to estimate numerical values of engineering characteristics of soils for use in low dams where adequate safety factors are provided.

In 1952 the Bureau of Reclamation and the Corps of Engineers, with Prof. Arthur Casagrande of Harvard University as consultant, reached agreement on a modification of Professor Casagrande's airfield classification which they named the "Unified Soil Classification System." This system, which is particularly applicable to the design and construction of dams, takes into account the engineering properties of soils, is descriptive and easy to associate with actual soils, and has the flexibility of being adaptable both to the field and to the laboratory. Probably its greatest advantage is that a soil can be classified readily by visual and manual examination without the necessity for laboratory testing. The Unified Soil Classification System is based on the size of the particles, the amounts of the various sizes, and the characteristics of the very fine grains.

A soil mass consists of solid particles and pore fluids. The solid particles generally are mineral grains of various sizes and shapes, occurring in every conceivable arrangement. These solid particles can be divided into various components, each of which contributes its share to the physical properties of the whole. Soil classification can best be understood by first considering the properties of these soil components. Accordingly, sections 85, 86, and 87 describe the constituents of soil and introduce concepts used in the system. Section 88

gives the essentials of the classification scheme for soils found in nature as shown in the Unified Soil Classification Chart. In addition to proper classification, it is important to include an adequate description of the soil in reports or logs of explorations. The classification chart contains information required for describing soils and includes examples. Additional information on soil descriptions is given in part H. Section 89 contains a comparison of the engineering properties of typical soils of each classification group.

85. ***Soil Components.***—(a) *Size.*—Particles larger than 3 inches are excluded from the Unified Soil Classification System. The amount of each oversized material, however, may be of great importance in the selection of sources for embankment material; hence, logs of exploration always contain information on quantity and size of particles larger than 3 inches. For definitions of terms for materials larger than 3 inches (cobbles, boulders, rock), see appendix D.

Within the size range of the system there are two major divisions; namely, the coarse grains and the fine grains. Coarse grains are those larger than the No. 200 sieve size (0.074 mm.), and they are further divided as follows:

Gravel (G), from 3 inches to No. 4 sieve (3/16 inch):
- Coarse gravel—3 inches to 3/4 inch.
- Fine gravel—3/4 inch to No. 4 sieve.

Sand (S), from No. 4 sieve to No. 200 sieve:
- Coarse sand—No. 4 to No. 10 sieve.
- Medium sand—No. 10 to No. 40 sieve.
- Fine sand—No. 40 to No. 200 sieve.

For visual classification, 1/4 inch is considered equivalent to the No. 4 sieve size, and the No. 200 sieve is about the smallest size of particles that can be distinguished individually by the unaided eye.

Fine grains or fines are smaller than the No. 200 sieve size and are of two types: *silt* (M) and *clay* (C). Older classification systems defined clay variously as those particles smaller than 5 microns (0.005 mm.) or 2 microns (0.002 mm.), and they defined silt as fines larger than the clay sizes. (See fig. 29.) It is a mistaken idea, however, that the typical engineering characteristics of silt and clay correspond to particular grain sizes. Natural deposits of rock flour that exhibit all the properties of silt and none of clay may consist entirely of grains smaller than 5 microns. On the other hand typical clays may consist mainly of particles larger than 5 microns but containing small quantities of extremely fine, colloidal-sized particles. Size distinction is not made between silt and clay in the Unified Soil Classification System; rather, the two materials are differentiated by their behavior.

Organic material (O) is often a component of soil, but it has no specific grain size. It is distinguished by the composition of its particles rather than by their sizes, which range from colloidal-sized particles of molecular dimensions to fibrous pieces of partly decomposed vegetable matter several inches in length.

(b) *Gradation.*—The amounts of the various sizes of grains present in a soil can be determined in the laboratory by means of sieving, for the coarse grains, and by sedimentation (wet mechanical analysis) for the fines, as described in section 115. The laboratory results are usually presented in the form of a cumulative grain-size curve. For soils consisting mainly of coarse grains, the grain-size distribution reveals something of the physical properties of the material. On the other hand the grain size is much less significant for soils containing a preponderance of fine grains.

Typical gradations of soils are:

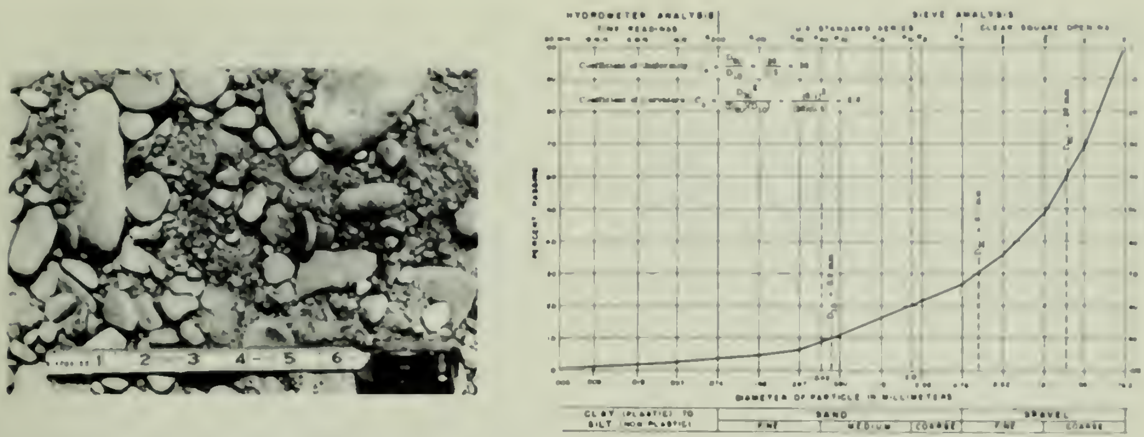
Well graded (W)—Good representation of all particle sizes from largest to smallest.

Poorly graded (P)—Uniform, most particles about the same size; or skip (or gap) gradation—absence of one or more intermediate sizes.

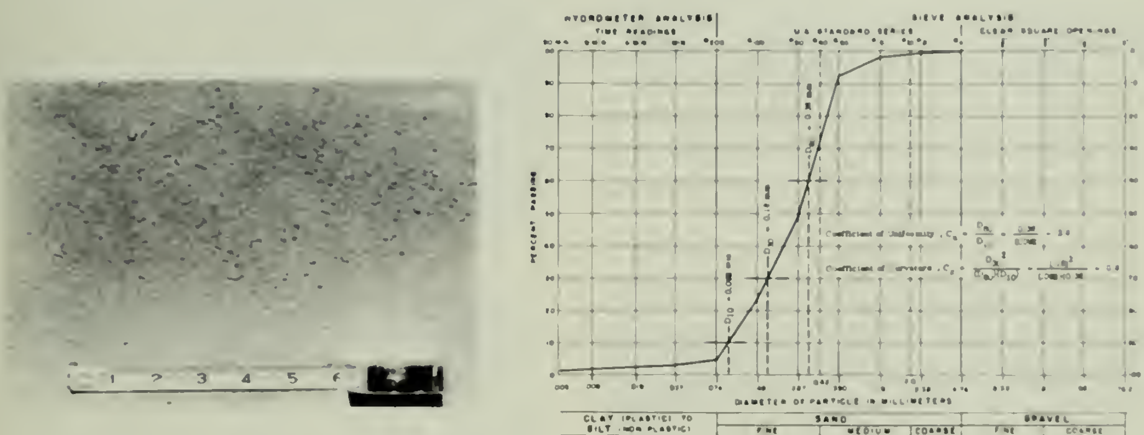
In the field, soil is estimated to be well graded or poorly graded by visual examination. For laboratory purposes the type of gradation can be determined by the use of criteria based on the range of sizes and on the shape of the grain-size curve. The measure of size range is called the coefficient of uniformity, C_u, which is the ratio of the 60-percent-finer-than size (D_{60}) to the 10-percent-finer-than size (D_{10}). The shape of the grain-size curve is given by the coefficient of curvature, C_c, which is the ratio of the square of the 30-percent-finer-than size $(D_{30})^2$ to the product of (D_{60}) by (D_{10}). Photographs of typical gradations and corresponding grain-size curves are shown in figure 32.

(c) *Shape.*—The shape of the particles has an important influence on the physical properties of a soil. The following shapes are most common:

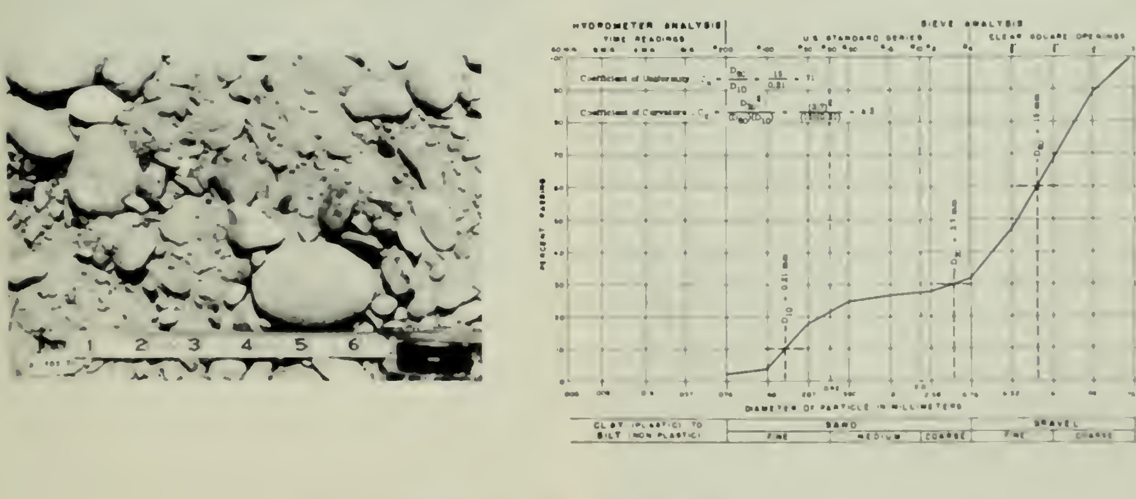
Bulky or equidimensional grains.—These may be further described as rounded, subrounded, subangular, and angular (fig. 33). The coarse-



(A) WELL-GRADED GRAVEL (GW), VERMEJO, PROJECT, NEW MEXICO



(B) UNIFORM SAND (SP), CHERRY CREEK RESERVOIR, COLORADO



(C) POORLY GRADED GRAVEL (GP), FALCON DAM, TEXAS

Figure 32. Typical soil gradations.

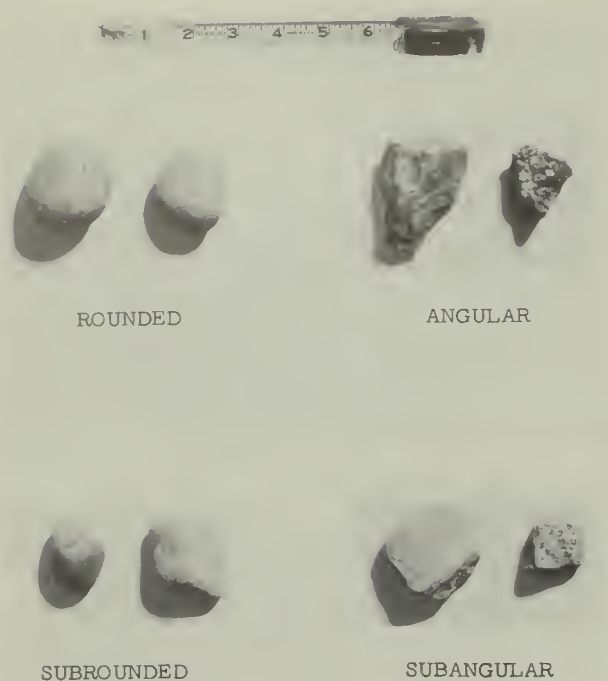


Figure 33. **Typical shapes of bulky grains.**

grained components of a soil are usually of the bulky type, consisting chiefly of the minerals quartz and feldspar.

Flaky grains, also called platelike particles.—These are present in appreciable quantities in many fine-grained soils. Mica and some clay minerals have this shape which is mainly responsible for their high compressibility (fig. 34 (A) and (B)).

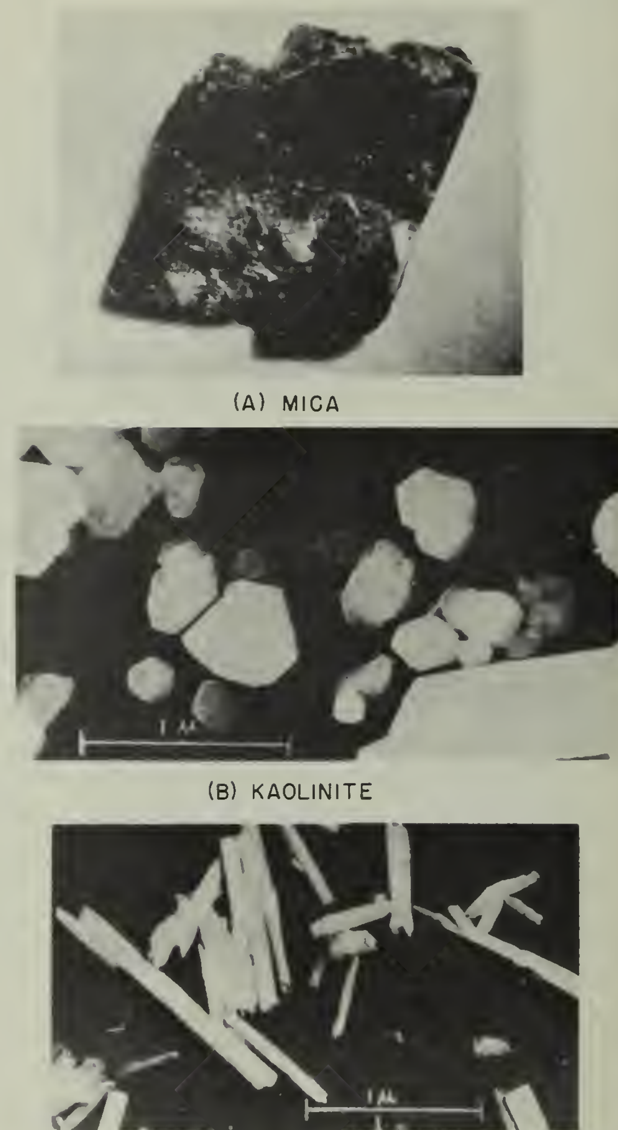
Elongated grains and fibers.—The most commonly encountered materials in this class are the clay mineral halloysite (fig. 34(C)), asbestos, some types of volcanic ash, and organic soils such as peat.

86. ***Soil Moisture.***—A typical soil mass has three constituents—soil grains, air, and water. In soils consisting largely of fine grains, the amount of water present in the voids has a pronounced effect on the soil properties. Three main states of soil consistency are recognizable:

(1) *Liquid state,* in which the soil is either in suspension or behaves like a viscous fluid;

(2) *Plastic state,* in which the soil can be rapidly deformed or molded without rebounding elastically, changing volume, cracking, or crumbling; and

(3) *Solid state,* in which the soil will crack when deformed or will exhibit elastic rebound.



(A) MICA

(B) KAOLINITE

(C) HALLOYSITE

Figure 34. **Flaky and elongated grains.**

In describing these soil states it is customary to consider only the fraction of soil smaller than the No. 40 sieve size (the upper limit of the fine sand component). For this soil fraction the water content in percentage of dry weight at which the soil passes from the liquid state into the plastic state is called the liquid limit (LL). A device (fig. 35) which causes the soil to flow under certain conditions is used in the laboratory to determine the liquid limit as described in section 115. Similarly, the water content of the soil at the boundary between the plastic state and

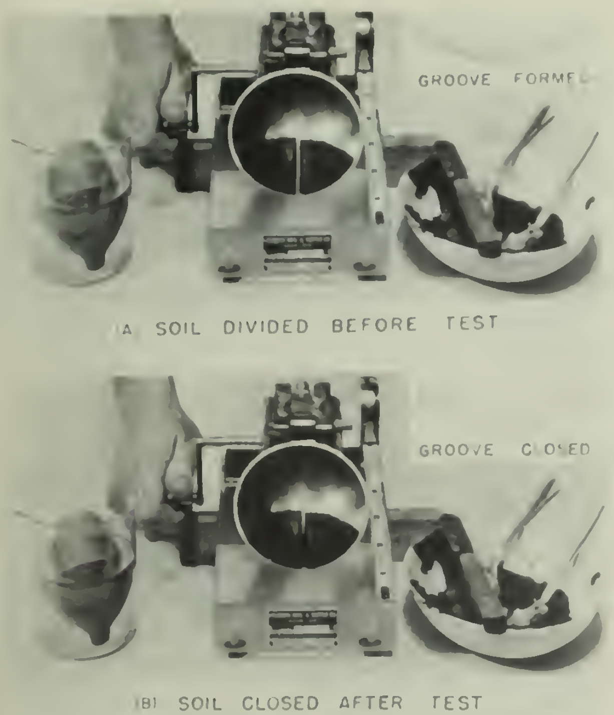


Figure 35. **Test for liquid limit.**

the solid state is called the plastic limit (PL). The laboratory test described in section 115 consists of repeatedly rolling threads of the soil to one-eighth-inch diameter until they crumble and then determining the water content (fig. 36). The difference between the liquid limit and the plastic limit corresponds to the range of water contents within which the soil is plastic. This difference of water content is called the plasticity index (PI). Highly plastic soils have high PI values. In a nonplastic soil the plastic limit and the liquid limit are the same and the PI equals 0.

These limits of consistency, which are called "Atterberg limits" after a Swedish scientist, are used in the Unified Soil Classification System as the basis for laboratory differentiation between materials of appreciable plasticity (clays) and slightly plastic or nonplastic materials (silts). With sufficient experience a soils engineer may acquire the ability to estimate the Atterberg limits of a soil. However, three simple hand tests have been found adequate for field identification and classification of fine soils and for determining whether the fine-grained fraction of a soil is silty or clayey, without requiring estimation of Atterberg limits. These hand tests, which are part of the field procedure in the Unified Soil Classification System, are as follows:

Dilatancy (reaction to shaking).

Dry strength (crushing characteristics).

Toughness (consistency near plastic limit).

They are discussed in the following section.

87. *Properties of Soil Components.* (a) *Gravel and Sand.* Both of the coarse-grained components of soil (gravel and sand) have essentially the same engineering properties, differing mainly in degree. The division of gravel and sand sizes by the No. 4 sieve is arbitrary and does not correspond to a sharp change in properties. Well-graded, compacted gravels or sands are stable materials. The coarse-grained soils when devoid of fines are pervious, easy to compact, little affected by moisture, and not subject to frost action. Although grain shape and gradation, as well as size, affect these properties, gravels are generally more pervious, more stable, and less affected by water or frost than are sands, for the same amount of fines.

As a sand becomes finer and more uniform, it approaches the characteristics of silt, with cor-

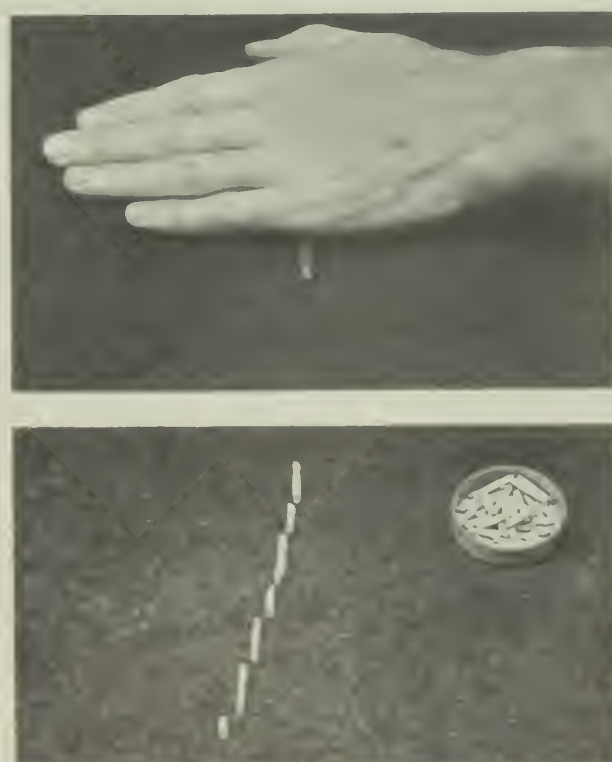


Figure 36. **Test for plastic limit.**

responding decrease in permeability and reduction in stability in the presence of water. Very fine, uniform sands are difficult to distinguish visually from silt. Dried sand, however, exhibits no cohesion (does not hold together) and feels gritty in contrast to the very slight cohesion and smooth feel of dried silt.

(b) *Silt and Clay.*—Even small amounts of fines may have important effects on engineering properties of the soils in which they are found. As little as 10 percent of particles smaller than the No. 200 sieve size in sand and gravel may make the soil virtually impervious, especially when the coarse grains are well-graded. Also, serious frost heaving in well-graded sands and gravels may be caused by less than 10 percent of fines. The utility of coarse-grained materials for surfacing roads can be improved by the addition of a small amount of clay to act as a binder for the sand and gravel particles.

Soils containing large quantities of silt and clay are the most troublesome to the engineer. These materials exhibit marked changes in physical properties with change of water content. A hard, dry clay, for example, may be suitable as a foundation for heavy loads so long as it remains dry, but may turn into a quagmire when wet. Many of the fine soils shrink on drying and expand on wetting, which may adversely affect structures founded upon them or constructed of them. Even when the water content does not change, the properties of fine soils may vary considerably between their natural condition in the ground and their state after being disturbed. Deposits of fine particles which have been subjected to loading in geologic time, frequently have a structure which gives the material unique properties in the undisturbed state. When the soil is excavated for use as a construction material or when the natural deposit is disturbed, for example by driving piles, the soil structure is destroyed and the properties of the soil are changed radically.

Silts are different from clays in many important respects, but because of similarity in appearance, they often have been mistaken one for the other, sometimes with unfortunate results. Dry, powdered silt and clay are indistinguishable, but they are easily identified by their behavior in the presence of water. Recognition of fines as being silt or clay is an essential part of the Unified Soil Classification System.

(A) REACTION TO SHAKING

(B) REACTION TO SQUEEZING

***Figure 37.* Dilatancy test for silt.**

Silts are the nonplastic fines. They are inherently unstable in the presence of water and have a tendency to become "quick" when saturated. Quick silts often are called "bull's liver" by construction men. Silts are fairly impervious, difficult to compact, and are highly susceptible to frost heaving. Silt masses undergo change of volume with change of shape (the property of dilatancy), in contrast to clays which retain their volume with change of shape (the property of plasticity). The dilatancy property, together with the "quick" reaction to vibration, affords a means of identifying typical silt in the loose, wet state. The dilatancy test is illustrated by the photograph of figure 37, and is described on the Classification Chart (fig. 38). When dry, silt can be pulverized easily under finger pressure (indicative of very slight dry strength), and will have a smooth feel between the fingers in contrast to the grittiness of fine sand.

Silts differ among themselves in size and shape of grains, which are reflected mainly in the prop-

ON

1 3

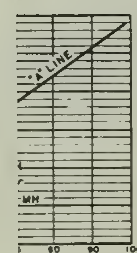
irements for GW

Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.

3

ements for SW

Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols



RAINED SOILS

n of
utty.
spread
hen the
s into
nd
s
ity,

slight

when it
oil.
e of the
ticity,
r below

nit.

103-D-347

erty of compressibility. For similar conditions of previous geologic loading, the higher the liquid limit of a silt, the more compressible it is. The liquid limit of a typical bulky-grained, inorganic silt is about 30 percent, while highly micaceous or diatomaceous silts (so-called elastic silts), consisting mainly of flaky grains, may have liquid limits as high as 100 percent. The differences in quicking and dilatancy properties afford a means of distinguishing in the field between silts of low liquid limits (L) and those of high liquid limits (H).

Clays are the plastic fines. They have low resistance to deformation when wet, but they dry to hard, cohesive masses. Clays are virtually impervious, difficult to compact when wet, and impossible to drain by ordinary means. Large expansion and contraction with changes in water content are characteristics of clays. The small size, flat shape, and type of mineral composition of clay particles combine to produce a material that is both compressible and plastic. The higher the liquid limit of a clay, the more compressible it will be when compared at equal conditions of previous geologic loading. Hence, in the Unified Soil Classification System, the liquid limit is used to distinguish between clays of high compressibility (H) and those of low compressibility (L). Differences in plasticity of clays are reflected by their plasticity indexes. At the same liquid limit, the higher the plasticity index, the more cohesive is the clay.

Field differentiation among clays is accomplished by the toughness test in which the moist soil is molded and rolled into threads until crumbling occurs, and by the dry strength test which measures the resistance of the clay to breaking and pulverizing. The dry strength and the toughness tests are shown in the Classification Chart (fig. 38). With a little experience in performing these tests, the clays of low compressibility and low plasticity, "lean" clays (L) can be readily distinguished from the highly plastic, highly compressible, "fat" clays (H).

(c) *Organic Matter.*—Organic matter in the form of partly decomposed vegetation is the primary constituent of peaty soils. Varying amounts of finely divided vegetable matter are found in plastic and in nonplastic sediments and often affect their properties sufficiently to influence their classification. Thus, we have organic silts and silt clays of low plasticity and organic clays of medium to high plasticity. Even small amounts of organic material in colloidal form in a clay will result in an appreciable increase in liquid limit of the material without increasing its plasticity index. Organic soils are dark gray or black in color and usually have a characteristic odor of decay. Organic clays feel spongy in the plastic range as compared to inorganic clays. The tendency for soils high in organic content to create voids by decay or to change the physical characteristics of a soil mass through chemical alteration makes them undesirable for engineering use. Soils containing even moderate amounts of organic matter are significantly more compressible and less stable than inorganic soils; hence, they are less desirable for engineering use.

88. *Unified Soil Classification System.* —(a) *General.* Soils in nature seldom exist separately as gravel, sand, silt, clay, or organic matter but are usually found as mixtures with varying proportions of these components. The Unified Soil Classification System is based on recognition of the type and predominance of the constituents, considering grain size, gradation, plasticity, and compressibility. It divides soils into three major divisions: coarse-grained soils, fine-grained soils, and highly organic (peaty) soils. In the field, identification is accomplished by visual examination for the coarse grains and by a few simple hand tests for the fine-grained soils or fraction. In the laboratory the grain-size curve and the Atterberg limits can be used. The peaty soils (Pt) are readily identified by color, odor, spongy feel, and fibrous texture, and are not further subdivided in the classification system.

(b) *Field Classification.*—A representative sample of soil (excluding particles larger than 3 inches) is first classified as coarse-grained or fine-grained by estimating whether 50 percent, by weight, of the particles can be seen individually by the naked eye. Soils containing more than 50 percent of particles that can be seen are coarse-grained soils; soils containing more than 50 percent of particles smaller than the eye can see are fine-grained soils. If the soil is predominantly coarse-grained, it is then identified as being a gravel or a sand by estimating whether 50 percent or more, by weight, of the coarse grains are larger or smaller than the No. 4 sieve size (about ¼ inch).

If the soil is a gravel, it is next identified as being "clean" (containing little or no fines), or "dirty" (containing an appreciable amount of

fines). For clean gravels final classification is made by estimating the gradation: the well-graded gravels belong to the GW group, and uniform and skip-graded gravels belong to the GP group. Dirty gravels are of two types: those with nonplastic (silty) fines (GM) and those with plastic (clayey) fines (GC). The determination of whether the fines are silty or clayey is made by the three manual tests for fine-grained soils.

If a soil is a sand the same steps and criteria are used as for the gravels in order to determine whether the soil is a well-graded clean sand (SW), poorly graded clean sand (SP), sand with silty fines (SM), or sand with clayey fines (SC).

If a material is predominantly (more than 50 percent by weight) fine-grained, it is classified into one of six groups (ML, CL, OL, MH, CH, OH) by estimating its dilatancy (reaction to shaking), dry strength (crushing characteristics), and toughness (consistency near the plastic limit), and by identifying it as being organic or inorganic. The test procedures and the behavior of the various groups of fine-grained soils for each of the hand tests are shown on the Classification Chart (fig. 38).

Soils that are typical of the various groups are readily classified by the foregoing procedures. Many natural soils, however, will have property characteristics of two groups, because they are close to the borderline between the groups either in percentages of the various sizes or in plasticity characteristics. For this substantial number of soils, boundary classifications are used; that is, the two group symbols most nearly describing the soil are connected by a hyphen, such as GW–GC.

If the percentages of gravel and sand sizes in a coarse-grained soil are nearly equal, the classification procedure is to assume that the soil is a gravel and then continue on the chart until the final soil group, say GC, is reached. Since it could have been assumed that the soil is a sand, the correct field classification is GC–SC, because the criteria for the gravel and sand subgroups are identical. Similarly, within the gravel or sand groupings, boundary classifications such as GW–GP, GM–GC, GW–GM, SW–SP, SM–SC, and SW–SM, can occur.

Proper boundary classification of a soil near the borderline between coarse-grained and fine-grained soils is accomplished by classifying it first as a coarse-grained soil and then as a fine-grained soil. Such classifications as SM–ML and SC–CL are common.

Within the fine-grained division, boundary classifications can occur between low-liquid-limit soils and high-liquid-limit soils as well as between silty and clayey materials in the same range of liquid limits. For example, one may find ML–MH, CL–CH, and OL–OH soils; ML–CL, ML–OL, and CL–OL soils; and MH–CH, MH–OH, and CH–OH soils.

(c) *Laboratory Classification.*—Although most classifications of soil will be done visually and by simple hand tests, the Unified Soil Classification System has provided for precise delineation of the soil groups by mechanical analyses and Atterberg limits tests in the laboratory. Laboratory classifications are often performed on representative samples of soils which are being subjected to extensive testing and to verify field classifications when used in the design of small dams. Laboratory classification can be used to advantage in training the field classifier of soils to improve his ability to estimate percentages and degrees of plasticity.

The grain-size curve is used to classify the soil as being coarse-grained or fine-grained, and if coarse-grained, into gravel or sand by size, using the 50-percent criterion. Within the gravel or sand groupings, soils containing less than 5 percent finer than the No. 200 sieve size are considered "clean" and are then classified as well graded or poorly graded by their coefficients of uniformity and of curvature. In order for a clean gravel to be well graded (GW), it must have *both* a coefficient of uniformity, C_u, greater than 4 and a coefficient of curvature, C_c, between 1 and 3; otherwise, it is classified as a poorly graded gravel (GP). A clean sand having *both* C_u greater than 6 and C_c between 1 and 3 is in the SW group; otherwise, it is a poorly graded sand (SP).

Laboratory classification criteria for coarse-grained soils and for fine-grained soils are given in the Soil Classification Chart, figure 38.

89. *Engineering Characteristics of Soil Groups.*—(a) *General.*—Although there is no satisfactory substitute for actual testing to determine the important engineering properties of a particular soil, approximate values for typical soils of each classification group can be given as a result of statistical analysis of existing information. The attempt to put soils data into quantitative form involves the

risk of (1) the data not being representative, and (2) use of the values in design without adequate safety factors. For the design of small dams, however, where investigation has disclosed no complex problems, expensive laboratory tests of permeability, shear, and consolidation of soils appear unwarranted and the use of average values of these properties is permissible. Since the values pertain to the soil groups, proper soil classification becomes of vital importance. Verification of field identification by laboratory gradation and Atterberg limits tests should be made on representative samples of each soil group encountered.

Table 6 is a summary of values obtained on more than 1,500 soil tests performed in the Engineering Laboratories of the Bureau of Reclamation in Denver, Colo., arranged according to the main soil classification groups and two frequently occurring boundary groups. The data for this table were obtained from reports for which laboratory soil classifications were available. The large majority of soils were from the 17 Western States of the United States in which the Bureau operates; however, some foreign soils were included. Although the sampling area of the soils tested is limited, it is believed that the Unified Soil Classification System is relatively insensitive to geographical distribution. The procedure for determining which of many submitted samples should be tested is in itself conducive to obtaining a representative range of values, since samples were selected from the coarsest, finest, and average soil within a potential source.

For each soil property listed, the average and its 90 percent confidence limits are given where sufficient data were available to determine them. Since all laboratory tests, except large-sized permeability tests, were made on the minus No. 4 fraction of the soil, data on average values for the gravels are not available for most properties. However, an indication as to whether these average values will be greater than or less than the average values for the corresponding sand group is given in the table. The averages shown are subject to uncertainties that arise from sampling fluctuations, and they tend to vary from the true averages more widely if the number of observations is small. The plus or minus limits given are determined mathematically from the number of observations and from the standard deviation of the data used to determine the average. These limits imply that the true average, obtained by securing and testing more and more samples under the same essential conditions, lies within the plus or minus values 9 chances out of 10 [4].

The values for Proctor maximum dry density and optimum water content were obtained by tests described in section 115. The other properties are based on tests made on samples compacted to Proctor maximum dry density at optimum water

TABLE 6.—*Average properties of soils*

Soil classification group	Proctor compaction		Void ratio, e_o	Permeability, k, feet per year	Compressibility		Shearing strength		
	Maximum dry density in pounds per cubic foot	Optimum water content, percent			@ 20 p.s.i., percent	@ 50 p.s.i., percent	C_o p.s.i.	C_{sat} p.s.i.	tan φ
GW	>119	<13.3	(*)	27,000±13,000	<1.4	(*)	(*)	(*)	>0.79
GP	>110	<12.4	(*)	64,000±34,000	<0.8	(*)	(*)	(*)	>0.74
GM	>114	<14.5	(*)	>0.3	<1.2	<3.0	(*)	(*)	>0.67
GC	>115	<14.7	(*)	>0.3	<1.2	<2.4	(*)	(*)	>0.60
SW	119±5	13.3±2.5	0.37±*	(*)	1.4±*	(*)	5.7±0.6	(*)	0.79±0.02
SP	110±2	12.4±1.0	0.50±0.03	>15.0	0.8±0.3	(*)	3.3±0.9	(*)	0.74±0.02
SM	114±1	14.5±0.4	0.48±0.02	7.5±4.8	1.2±0.1	3.0±0.4	7.4±0.9	2.9±1.0	0.67±0.02
SM-SC	119±1	12.8±0.5	0.41±0.02	0.8±0.6	1.4±0.3	2.9±1.0	7.3±3.1	2.1±0.8	0.66±0.07
SC	115±1	14.7±0.4	0.48±0.01	0.3±0.2	1.2±0.2	2.4±0.5	10.9±2.2	1.6±0.9	0.60±0.07
ML	103±1	19.2±0.7	0.63±0.02	0.59±0.23	1.5±0.2	2.6±0.3	9.7±1.5	1.3±*	0.62±0.04
ML-CL	109±2	16.8±0.7	0.54±0.03	0.13±0.07	1.0±0.2	2.2±0.0	9.2±2.4	3.2±*	0.62±0.06
CL	108±1	17.3±0.3	0.56±0.01	0.08±0.03	1.4±0.2	2.6±0.4	12.6±1.5	1.9±0.3	0.54±0.04
OL	(*)	(*)	(*)	(*)	(*)	(*)	(*)	(*)	(*)
MH	82±4	36.3±3.2	1.15±0.12	0.16±0.10	2.0±1.2	3.8±0.8	10.5±4.3	2.9±1.3	0.47±0.05
CH	94±2	25.5±1.2	0.80±0.04	0.05±0.05	2.6±1.3	3.9±1.5	14.9±4.9	1.6±0.86	0.35±0.09
OH	(*)	(*)	(*)	(*)	(*)	(*)	(*)	(*)	(*)

The ± entry indicates 90 percent confidence limits of the average value. * Denotes insufficient data, > is greater than, < is less than

content. The value of void ratio, e_o, is the ratio of the portion of the volume of the soil mass occupied by water and air, to the volume of the soil grains. It is derived from the Proctor maximum dry density and the specific gravity of the grains. The MH and CH soil groups have no upper boundary of liquid limits in the classification; hence, it is necessary to give the range of those soils included in the table. The maximum liquid limits for the MH and the CH soils tested were 81 and 88 percent, respectively. Soils with higher liquid limits than these will have inferior engineering properties.

(b) *Permeability.*—The voids in the soil mass provide passages through which water may move. Such passages are variable in size and the paths of flow are tortuous and interconnected. If, however, a sufficiently large number of paths of flow are considered as acting together, an average rate of flow for the soil mass can be determined under controlled conditions that will represent a property of the soil. The water movement is called percolation; the measure of it is called permeability; and the factor relating permeability to unit conditions is called the coefficient of permeability, k, which represents the discharge through a unit area at unit hydraulic gradient. The use of k in estimating flow through soils is discussed in section 125(b). There are many units of measurement in common use for expressing the coefficient of permeability. The one used in table 6 is feet per year, or cubic feet per square foot per year at unit gradient. One foot per year is virtually equal to 10^{-6} centimeters per second.

The coefficient of permeability of natural soil deposits ranges from 1 million feet to 0.001 foot per year. In many soil deposits the permeability parallel to the bedding planes may be 100 or even 1,000 times as large as the permeability at right angles to the bedding planes. Permeability in some soils is very sensitive to small changes in density, water content, or gradation. Because of the possible wide variation in permeability, a numerical value of k should be considered only as an order of magnitude. It is customary to describe soils with permeabilities less than 1 foot per year as impervious; those with permeabilities between 1 and 100 feet per year as semipervious; and soils with permeabilities greater than 100 feet per year as pervious. These values, however, are not absolute for the design of dams. Successful structures have been built whose various zones were constructed of soils with permeabilities not within these respective ranges.

(c) *Compressibility.*—Two values are given for compressibility: the value at 20 pounds per square inch effective stress, and the value at 50 pounds per square inch effective stress. These values are for confined compression with drainage permitted. In the confined compression test the soil is prevented from moving laterally by the sides of the container. Porous stones on the top and the bottom permit the water and air in the compacted specimens to drain under the load. The value recorded is the percentage reduction of initial volume at equilibrium under the applied vertical stress. The phenomenon of compressibility is associated with changes in volume in the voids and only to a very limited extent with changes in the solid particles. If the voids are to a large extent filled with air, the addition of a load on the soil mass will result in compression almost immediately. If, on the other hand, the voids are very nearly or completely filled with water, very little or no compression will take place immediately upon application of the load, and only as the water drains from the soil mass will consolidation take place. If the water can drain readily from the soil mass, consolidation may take place in a relatively short period of time, but if the soil is very impervious and the soil mass is large, complete consolidation may require many years.

(d) *Shearing Strength.*—Three different values are given for the soil groups under this heading: C_o, C_{sat}, and tan ϕ. The values of C_o and tan ϕ are the vertical intercept and the slope, respectively, of the Mohr strength envelope on an effective stress basis. The Mohr plot is shown in figure 39. The Mohr strength envelope is obtained by testing several sealed specimens of soil, at the Proctor maximum dry density and optimum water content, in a triaxial shear machine in which pore-water pressures developed during the test are measured. The effective stresses are obtained by subtracting the measured pore-water pressures in the specimen from the stresses applied by the machine. No drainage is permitted during the tests; hence, they are sometimes called unconsolidated quick tests. The value C_{sat} was obtained by preparing a specimen at Proctor maximum dry density and optimum water content, saturating it, and shearing it to failure to obtain the small circle shown in figure 39. The value C_{sat}

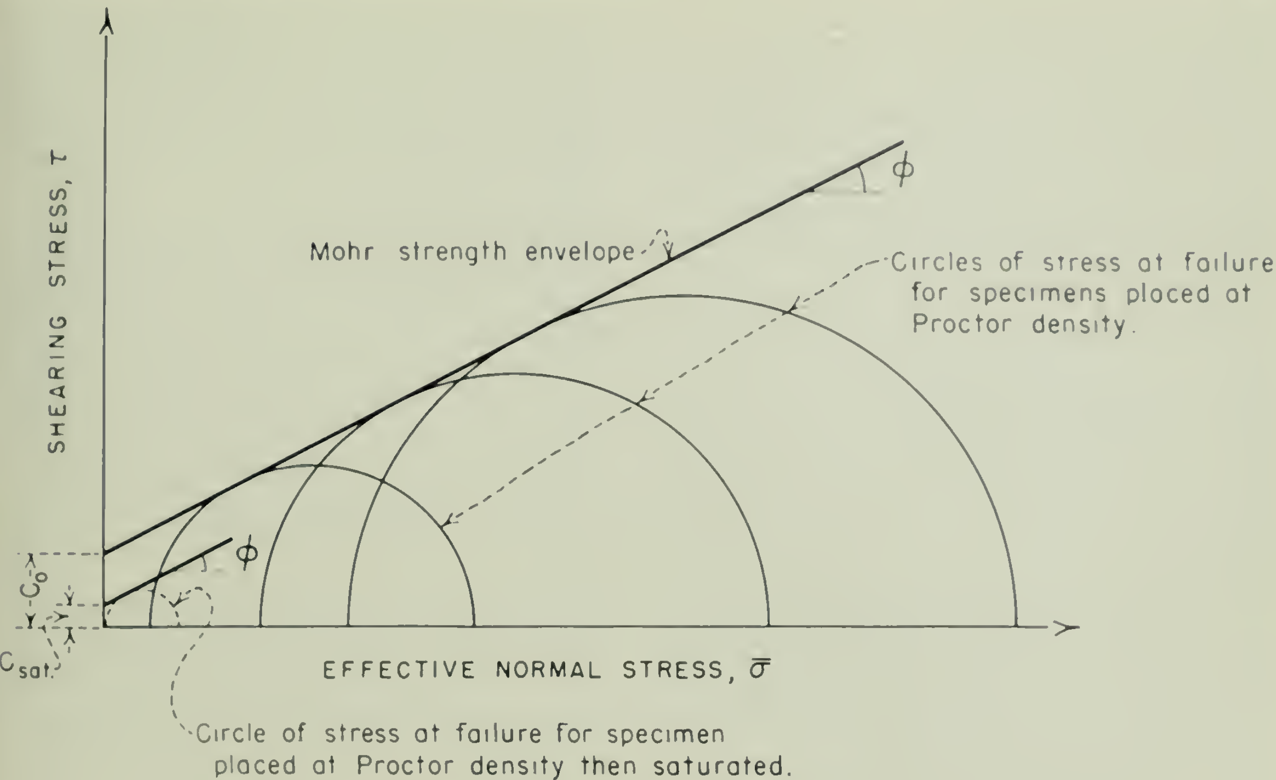


Figure 39. **Shearing strength of compacted soils.**

is the intercept on the vertical axis of a line tangent to the circle having an inclination ϕ.

These values for shearing strength are applicable for use in Coulomb's equation:

$$s = C + (\sigma - u) \tan \phi$$

where:

s = shearing strength per unit of area,
u = pore-water pressure,
σ = applied normal stress,
$\tan \phi$ is as previously defined, and
C is either C_o or C_{sat} depending on the water content of the soil.

A discussion of the significance of pore-water pressure in the laboratory tests is beyond the scope of this text. The effective-stress principle, however, which takes the pore-water pressures into account, was used in arriving at recommended slopes given in chapter V.

D. ROCK CLASSIFICATION

(Adapted from the Army publication Geology and its Military Applications [5])

90. *Rocks and Minerals.*—(a) *Definition and Types.*—In a broad sense rocks are aggregates of minerals. The principal exceptions to this definition are the products of organic decay such as coal, and volcanic glasses such as obsidian. To the engineer the term "rock" signifies firm and coherent or consolidated substances that cannot normally be excavated by manual methods alone. Based on the principal mode of origin, rocks are grouped into three large classes: igneous, sedimentary, and metamorphic. These are discussed in more detail in sections 91, 92, and 93.

(b) *Mineral Identification.*—The physical properties characteristic of a mineral, controlled by its chemical composition and molecular structure, are valuable aids in its rapid field identification. Those characteristics which can be determined by simple field tests are introduced to aid in the

identification of minerals and indirectly in the identification of rocks.

Hardness.—The hardness of a mineral is a measure of its ability to resist abrasion or scratching. A simple scale based on empirical tests for hardness has been universally accepted. The 10 minerals selected to form the standard of comparison, listed in order of increasing hardness from 1 to 10 are:

Talc or mica	1
Gypsum (fingernail about 2)	2
Calcite	3
Fluorite (copper coin between 3 and 4)	4
Apatite (knife blade about 5)	5
Feldspar (window glass about 5.5)	6
Quartz	7
Topaz or beryl	8
Corundum	9
Diamond	10

When testing the hardness of a mineral always use a fresh surface; always rub the mark to make sure it is really a groove made by scratching. If an unknown mineral scratches and in turn is scratched by a member of the scale or a testing medium (copper coin, pocketknife, or window glass), the two are of equal hardness.

Cleavage.—A material is said to have cleavage if smooth, plane surfaces are produced when the mineral is broken. This is a fairly consistent physical property of minerals and when present is of great value in their identification. Cleavage invariably occurs along parallel planes. Some minerals have one cleavage; others have two, three, or even more different cleavage directions which may have varying degrees of eminence. The number of cleavage directions and the angle at which they intersect serve as aids in identification of a mineral (fig. 40).

Fracture.—The broken surface of a mineral, in directions other than those of cleavage planes, is called the fracture. In some cases this property may be very helpful in field identification. The common types of fracture are conchoidal, if the fracture has concentric curved surfaces like the inside of a clamshell; irregular, if the surface is rough; and splintery if it has the appearance of wood.

Figure 40. Mineral cleavage. (U.S. Corps of Engineers.)

Luster.—The luster of a mineral is the appearance of its surface due to the quality and intensity of the light reflected. Two major kinds are recognized, metallic and nonmetallic. The main difference between the two is indicated by the name. In addition metallic minerals are opaque, or nearly so, whereas nonmetallic minerals are transparent on their thin edges. Some of the common nonmetallic lusters are vitreous, having the luster of glass; pearly, having the irridescence of pearl; and adamantine, having brilliant luster like that of a diamond.

Color.—The color of a mineral, as an aid in its identification, must be used with proper precaution, since some show a wide range without perceptible change in composition. Color on the whole, however, is fairly consistent, particularly in the metallic minerals where it is a great help in field identification.

Streak.—The color of the fine powder of a mineral, obtained by rubbing it on some white substance, preferably unglazed porcelain, is known as its streak. The streak of a mineral is quite consistent within a given range, even though its color may vary.

(c) *Common Rock-Forming Minerals.*—Only a dozen or so of the 2,000 known varieties of minerals are found in most common rocks. Descriptions of the most important of these rock-forming minerals or mineral groups follow:

Quartz.—Silicon dioxide. Hardness, 7, scratches glass easily. No cleavage. Fracture conchoidal. Luster, vitreous. Common varieties, usually white or colorless. Streak, white or colorless. Typical examples are milky quartz and rock crystal quartz.

Feldspar group.—Potassium-aluminum silicate or sodium-calcium-aluminum silicate. Hardness, 6, scratches glass with difficulty. Luster, vitreous. Streak, white. Orthoclase is a common potassium-rich variety which is typically colorless, white, gray, pink, or red, and has two good directions of cleavage that intersect at 90° to each other (① of fig. 40). The sodium-calcium-rich feldspars, commonly referred to as plagioclase feldspar, are

typically of various shades of gray, have two cleavage directions that intersect at angles of nearly 90° to each other, and can be distinguished from orthoclase feldspar by the presence of fine, parallel lines that appear on the basal cleavage surface.

Mica group.—Complex potassium-aluminum silicates, often with magnesium, iron and sodium. Hardness, 2 to 3, can be scratched with the thumbnail. Good cleavage in one direction. Luster, vitreous to pearly. Transparent, with varying shades of yellow, brown, green, red, and black in thicker specimens. Streak, white. The true characteristic of this group is that its minerals are capable of being split very easily into extremely thin and flexible sheets. Biotites (black) and muscovites (white) are two representative varieties.

Amphibole group.—Complex calcium-magnesium-iron silicates. Hardness, 5 to 6. Cleavage in two directions at angles 56° and 124°. Color, light to dark green to black. Streak, white to grayish-green. Hornblende is a common variety that is usually distinguishable from other amphiboles by its dark color.

Pyroxene group.—Complex calcium-iron silicates, closely analogous chemically to the amphibole group. Hardness, 5 to 6. Two directions of cleavage, making angles of about 87° and 93°, an important characteristic useful in distinguishing between the minerals of the pyroxene and amphibole groups. Color, light to dark green to black. Streak, white to grayish-green. Augite is a common variety that can be distinguished from hornblende by its cleavage angles.

Olivine.—Magnesium-iron silicate. Hardness, 6.5 to 7. No cleavage. Luster, vitreous. Color, olive to grayish-green to brown. Streak, white to colorless. An important characteristic of this mineral is its friability or tendency to crumble into small grains, which is due to its granular texture.

Calcite and dolomite.—Calcium carbonate and calcium-magnesium carbonate. Hardness, 3 and 3.5 to 4. Perfect cleavage in three directions (② in fig. 40). Luster, vitreous to pearl. Usually white or colorless, but may appear in shades of gray, red, green, blue, or yellow. Streak, white. Calcite may develop in large crystals, whereas dolomite is commonly found in coarse, granular masses.

Clay minerals.—Extremely complex hydrous aluminum silicate. Hardness, 2 to 2.5. Luster, dull to earthy. Color, white, gray, greenish, and yellowish-white. The three most important groups of clay minerals are kaolinite, montmorillonite, and illite. Almost all clays contain one or more of these three groups. Clay minerals can be distinguished only under the microscope and with the aid of X-ray equipment. They occur typically in very fine-grained masses of thin micalike scales.

Limonite and hematite.—Hydrous ferric oxide and ferric oxide. Hardness, 5.5 and 6.5. No cleavage. Color, dark brown to black and reddish-brown to black, depending on the variety. Limonite has a yellowish-brown streak and is characteristically found in dark brown, nodular, earthy masses with no apparent crystal structure. Hematite has a light to dark Indian-red streak, usually occurs in earthy masses, but occasionally is found in botryoidal or reniform shapes known as kidney ore and in foliated masses known as specular iron. Limonite and hematite are important coloring and cementing minerals in many different rocks, especially in the sedimentary group.

91. *Igneous Rocks.*—(a) *General.*—The igneous rocks are commonly referred to as primary rocks. They are those rocks which have solidified from a molten mass called magma when in the body of the earth (intrusive rocks), or from lava when extruded on the earth's surface (extrusive rocks). Igneous rocks owe their variation in significant characteristics to differences in chemical composition of the original molten mass and to differences in physical conditions under which the molten mass solidified.

Intrusive igneous rock masses are shown in figure 41. Dikes are tabular igneous bodies that are commonly intruded at an angle to the bedding of the surrounding formation (fig. 42). Sills are

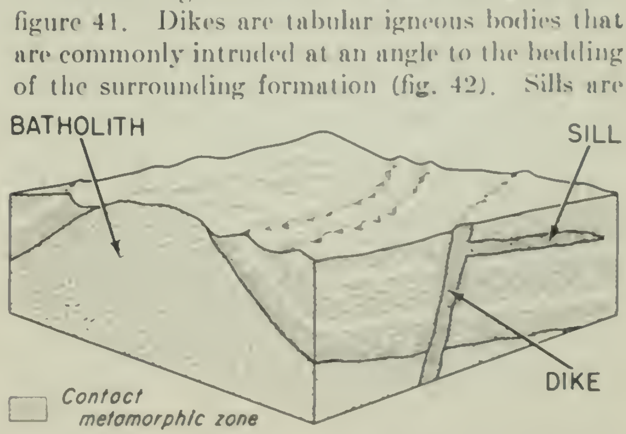


Figure 41. Intrusive igneous masses. (U.S. Corps of Engineers.)

Figure 42. **Three dikes cutting sedimentary beds. (U.S. Corps of Engineers.)**

similar bodies which are usually intruded parallel to the bedding planes of the rocks which enclose them. The thickness of a dike or a sill may vary from a few inches to several hundreds of feet, but this dimension is usually quite small in relation to the length and width of the intrusive body. For example, the Palisades sill of New York has a thickness of 1,000 feet and a length of over 100 miles. Very large, irregular masses of intrusive igneous rock covering many square miles of area are called batholiths. Although originally deeply buried beneath the earth's surface, they have become exposed through a process of uplift and erosion. A very striking example of an exposed batholith is the one in central Idaho which has an estimated area of over 80,000 square miles.

Extrusive igneous rock masses include lava flows and volcanic ejecta. Lava flows are the result of the solidification of lava which has issued from fissures in the earth's crust or poured out of volcanoes. These flows are the most common modes of occurrence of extrusive igneous rocks. Among the most notable of the enormous lava flows in the world is the Columbia River Plateau of Washington, Oregon, and Idaho. The lava sheets cover approximately 200,000 square miles and the succession of flows has a known cumulative thickness of 4,000 feet. Explosive volcanoes frequently eject great quantities of broken and pulverized rock material and blobs of molten lava which solidify before striking the ground. These solid volcanic ejecta are termed pyroclastic material, which varies in size from great blocks weighing many tons through small cinders or lapilli to fine dust-sized particles referred to as ash. (See fig. 43.)

(b) *Classification of Igneous Rocks.*—Chemical composition and texture are used to classify igneous rocks. A mass of molten rock material may be regarded as a complex solution containing oxide of silicon which behaves as an acid; and oxides of iron, aluminum, calcium, magnesium, potassium, and sodium which behave as bases. If more acid is available than is necessary to satisfy the bases in the magma, the surplus will show itself as free silicon dioxide (quartz), and the resulting rock is said to be acidic. If the bases are excessive, iron-magnesium minerals will be present and the rock is said to be basic. As a rule acidic rocks are light colored; basic rocks are dark to black. The one striking exception to this rule is obsidian, an acid rock which is normally black.

Texture refers to the size and arrangement of the mineral grains in the rock (fig. 44). These factors are influenced primarily by the rate at which the molten mass, magma or lava, cools. A constant rate of cooling produces rocks in which the constituent mineral grains are approximately the same size. In general the slower the molten material cools, the larger the size of the mineral grains. A change in the rate of cooling from an initial slow phase, followed by a more rapid phase, usually produces porphyritic texture (④ of fig. 44). These rocks are characterized by mineral grains of two dominant size groups—phenocrysts, or large grains, in a ground mass or background of smaller grains. Textural terms used in the classification of igneous rocks are:

Coarse-grained in which the crystals are visible to the naked eye (① of fig. 44).

Fine-grained in which crystals of like or equal size generally form throughout the whole mass of rock, but the individual grains can generally be seen only with a strong hand lens or with a microscope (② of fig. 44).

Glassy (noncrystalline) in which the rock has a noncrystalline or glassy texture (③ of fig. 44).

Table 7 lists the common igneous rocks. Those of similar chemical composition or mineral content are listed in the vertical column; those of similar

Figure 43. **Blocky type of solidified lava flows. Layer of volcanic ejecta (ash) covers area at left and in the foreground. (U.S. Corps of Engineers.)**

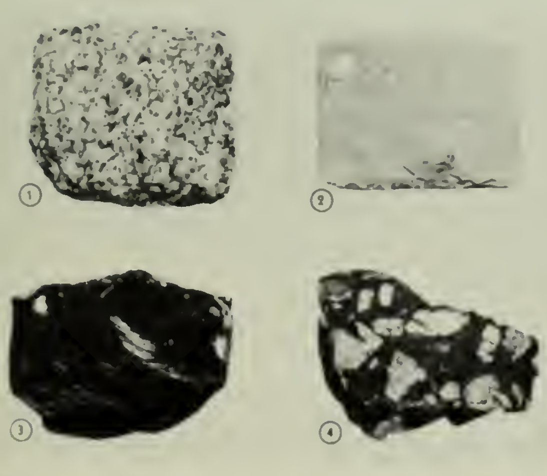


Figure 44. **Textures of igneous rocks: ① coarse-grained, ② fine-grained, ③ glassy, and ④ porphyritic. (U.S. Corps of Engineers.)**

textures are listed in the horizontal column. The following are the minerals in common igneous rocks:

Granite and *rhyolite* are composed largely of quartz and feldspar (mainly of the orthoclase variety), and as a rule contain mica (generally the biotite variety).

Diorite and *andesite* are composed of feldspar (mainly plagioclase varieties) and one or more dark minerals (biotite, hornblende, or pyroxene).

Gabbro and *basalt* differ from diorite in that the dark minerals (hornblende, pyroxene, and olivine) predominate. All feldspar is plagioclase; and biotites, although present in some gabbros, is distinctly uncommon.

Obsidian and *pitchstone* correspond in composition to granite and rhyolite. Both are commonly referred to as volcanic glasses. Obsidian is dark-colored to black and with a brilliant luster (③ of fig. 44). Pitchstone is lighter colored and with a dull luster.

Pumice is a porous or cellular glass, usually white or gray in color.

(c) *Primary Structural Features of Igneous*

TABLE 7.—*Common igneous rocks*

Texture	Composition		
	Acid rocks (more than 50 percent silica)		Basic rocks (less than 50 percent silica)
	Light-colored minerals, chiefly feldspar, predominate		Dark-colored minerals predominate
	Abundant quartz	Little or no quartz	No quartz—abundant amphibole, pyroxene, and plagioclase feldspar
Coarse-grained (mineral crystals easily visible to naked eye).	Granite	Diorite	Gabbro.
Fine-grained (mineral crystals generally invisible to naked eye).	Rhyolite	Andesite (Trap).	Basalt (Trap).
Glassy	Obsidian, pitchstone pumice.		

Rocks.—With the exception of those varieties which exhibit a glassy texture, igneous rocks are composed of interlocking grains of different minerals. On this basis they can be distinguished from crystalline sedimentary and massive metamorphic rocks which normally contain crystals of the same mineral. The distinctive structural features common to some, but not all, igneous rocks are as follows:

Flow structure may be exhibited by the glassy-textured igneous rocks, such as obsidian, and by the fine-grained extrusives, such as rhyolite.

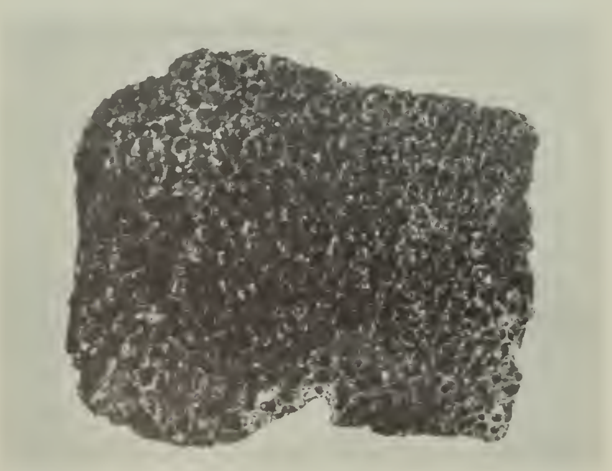


Figure 45. Scoriaceous structure in extrusive lava rock. (U.S. Corps of Engineers.)

Vesicular or *scoriaceous structure* is commonly present in extrusive igneous rocks (fig. 45). Such rock contains tiny spherical to almond-shaped openings called vesicles formed by gas bubbles in or rising through the lava.

Lamellar or *platy structure,* more or less perfect, may be found in some of the coarser grained igneous rocks. This structure is due to the parallel orientation of such minerals as mica and hornblende, and most commonly occurs near the contacts of intrusive bodies where the friction between the wall rock and the molten material causes the platy minerals to aline themselves in the direction of flow.

Columnar structure is often formed in fine-grained igneous rocks by the development of shrinkage cracks (joints) as the molten mass cools and solidifies. This structural feature is commonly found in basaltic intrusions, such as dikes and sills, that cooled at a moderate rate (fig. 46) and in many lava flows.

92. *Sedimentary Rocks.*—(a) *General.*—Sedimentary rocks, also known as stratified rocks, are of secondary origin. They are formed of layerlike masses of sediment that have hardened through cementation, compaction, or incipient recrystallization. The inorganic material entering into the composition of most sedimentary rocks is derived from the disintegration and decomposition of preexistent igneous, sedimentary, and metamorphic rocks. This material is then moved from its original position by water, wind, or glaciers in the form of solid particles or dissolved salts. Rock particles dropped from suspension produce deposits of clastic or fragmental sediment. By chemical reaction the dissolved salts become insoluble and form precipitated sediments; or by evaporation of the water medium they form evaporites.

The relative magnitude of pyroclastic deposits is not too well known, but it appears that they constitute only a small part of the sedimentary rocks in the earth's crust. The quantity of material, however, which can be ejected at a single volcanic eruption and transported by the wind is quite large. The organic material entering into the composition of a very small percentage of the total sedimentary rock mass is the result of the activities of plants and animals, either directly or indirectly. Included in this group are certain protective and supporting structures produced by

Figure 46. **Columnar jointing in basaltic sill. Upper photograph is a side view of sill showing regular arrangement of columns and lower photograph is a sectional view showing irregular hexagonal outline of each column. (U.S. Corps of Engineers.)**

plants and animals which on the death of the organism become sediments and certain precipitated sediments formed by the activities of organisms.

Based on the mode of origin, sediments can be classified as clastic, chemical, and organic. The clastic or fragmental sediments include gravel, sand, silt, and clay, which are differentiated by the dimensions of the particles and their plasticity characteristics. All kinds of rock contribute to clastic material. Each size of clastic particle may be transported by several agencies. The terms "gravel," "sand," "silt," and "clay" have been defined in section 85. The chemically deposited and organic sediments are classified on the basis of chemical composition. The common sediments formed chemically by precipitation and evaporation and through the life processes of organisms are listed in table 8.

The conversion of sediment into rock, sometimes called lithification, is brought about by a combination of the following processes:

Compaction, in which the rock or mineral particles are brought closer together by pressure of overlying materials such as conversion of clay to shale and conversion of peat to coal.

Cementation, in which porous materials are bound together by minerals precipitated from water solution such as silicon dioxide (quartz), calcium carbonate (calcite), and the iron oxides (limonite and hematite).

Recrystallization, in which a rock with an interlocking crystalline fabric or grain, such as crystalline limestone, is developed by continued growth of the mineral grains in a sediment or the development of new minerals from water.

(b) *Classification of Sedimentary Rocks.*—Sedimentary rocks may be classified as clastic, chemical, or organic, based on the mode of origin of the sediment from which they are derived. The clastic rocks commonly show separate grains. The chemical precipitates and evaporites, on the other hand, either have interlocking crystals or are in earthy masses. The organically formed rocks commonly contain easily recognized animal and plant remains, such as shells, bones, stems, or leaves. Table 8 lists the common sedimentary

TABLE 8.—*Common sedimentary rocks*

Type	Sediment	Rock
Clastic or fragmental	Coarse (gravel)	Conglomerate.
	Medium (sand)	Sandstone
	Fines (silt and clay)	Siltstone and shale
Pyroclastic	Coarse (cinder)	Agglomerate
	Fine (ash)	Tuff
Chemical precipitates and evaporites.	Calcium carbonate ($CaCO_3$)	Limestone
	Calcium magnesium carbonate ($Ca(Mg,Fe)(CO_3)_2$)	Dolomite
	Silicon dioxide (SiO_2)	Chert
	Calcium sulfate ($CaSO_4 \cdot 2H_2O$) ($CaSO_4$)	Gypsum, anhydrite.
	Sodium chloride (NaCl)	Rock salt
Organic	Calcium carbonate (animal remains)	Coquina, some coral rock, and chalk
	Carbon (plant remains)	Coal

rocks and the sediment or material from which they have been derived.

To analyze their mineral content, sedimentary rocks are divided into three major types: Sand, clay, and lime. The sand and clay types are principally clastic. The lime type includes the precipitates and the calcium carbonates of organic origin.

Sand-type rocks.—The minerals which are commonly found in the predominantly sand-type rocks such as conglomerates (fig. 47) and sandstone, are quartz as grains; feldspar as grains; mica minerals as small plates; clay minerals; and limonite, hematite, calcite, and quartz as cementing material.

Clay-type rocks.—The minerals which are commonly found in predominantly clay-type rocks, such as shale and siltstone, are clay minerals; quartz as fine grains; mica minerals as fine plates; and limonite, hematite, calcite, and quartz as cementing materials.

Lime-type rocks.—The minerals commonly found in the predominantly lime-type rocks, such as limestone, chalk, coral rocks, dolomite, and coquina, are calcite as visible grains or crystals; dolomite as visible grains or crystals; quartz as grains; chalcedony or chert as grains; clay minerals; and lime mud, limonite, hematite, and quartz as cementing materials.

(c) *Primary Structural Features of Sedimentary Rocks.*—The primary structural features inherent in the sediment before consolidation are valuable in the field recognition of sedimentary rocks. A universally prevalent structural feature of sedimentary rocks is their *stratification,* as indicated by differences in composition, texture, hardness, or color disposed in approximately parallel bands. These strata may be flat lying, or nearly so, as originally deposited; or they may be tilted or folded as a result of movement within the earth's crust. Each stratum is separated from the one immediately above and below by bedding planes or planes of stratification. The thicknesses of sedimentary strata vary from a few inches (thin bedded) to a few feet (thick bedded). Very thin beds are referred to as laminae.

A primary cleavage structure developed parallel to the stratification of some fine-grained sedimentary rocks is called *bedding* fissility. The ability of these rocks to split along parallel planes is attributed mainly to compositional and grain-size variations between layers. Shale is a sedi-

Figure 47. Conglomerate. (U.S. Corps of Engineers.)

mentary rock which has bedding fissility. Some sedimentary deposits, usually those composed of granular materials such as sand, commonly exhibit laminae lying at an angle to the true bedding plane. This feature of sedimentary rock is known as *crossbedding* or cross lamination.

Sediment deposited in low, flat places, such as flood plains of rivers and intermittent lakes, usually develops *mud cracks* which separate the mass into irregular polygonal blocks and which may become sufficiently hardened to be preserved during lithification of the sediment. Parallel ridges, known as *ripple marks*, developed in sediment moved by wind or water are often preserved when the sediment is consolidated. *Fossils*, the remains or impressions of animals and plants, are not structural features, but they are important to the field identification of sedimentary rocks. (See fig. 48.)

93. *Metamorphic Rocks.* (a) *General.* Metamorphic rocks are those formed from preexistent igneous or sedimentary rocks as a result of an enforced adjustment of these rocks to environments different from those in which they were originally formed. This adjustment may include the formation within the rock of new structures, textures, or minerals, or all of these.

Temperature, pressure, and chemically active fluids and gases are the major interrelated factors involved in metamorphism. Each factor is capable of accomplishing metamorphic work individually as follows:

Temperature. The effect of heat is twofold. It increases the solvent action of fluids, and it helps break up and change chemical compounds. Extremely high temperatures may result from the intrusion of molten masses. The zone of altered rock formed adjacent to the molten mass is called the contact metamorphic zone (fig. 41). Heat may also be a normal complement to the depth to which the rocks are buried. In this case the earth's own heat produces metamorphism, and the process is called geothermal metamorphism.

Pressure. The compressive forces which accompany mountain building and other disturbances in the earth's crust are in the main responsible for the pressures to which many rocks are subjected. By the action of these movements rocks could be produced in which the crystals, grains, and rock fragments are flattened and elongated or pulverized as a result of the force.

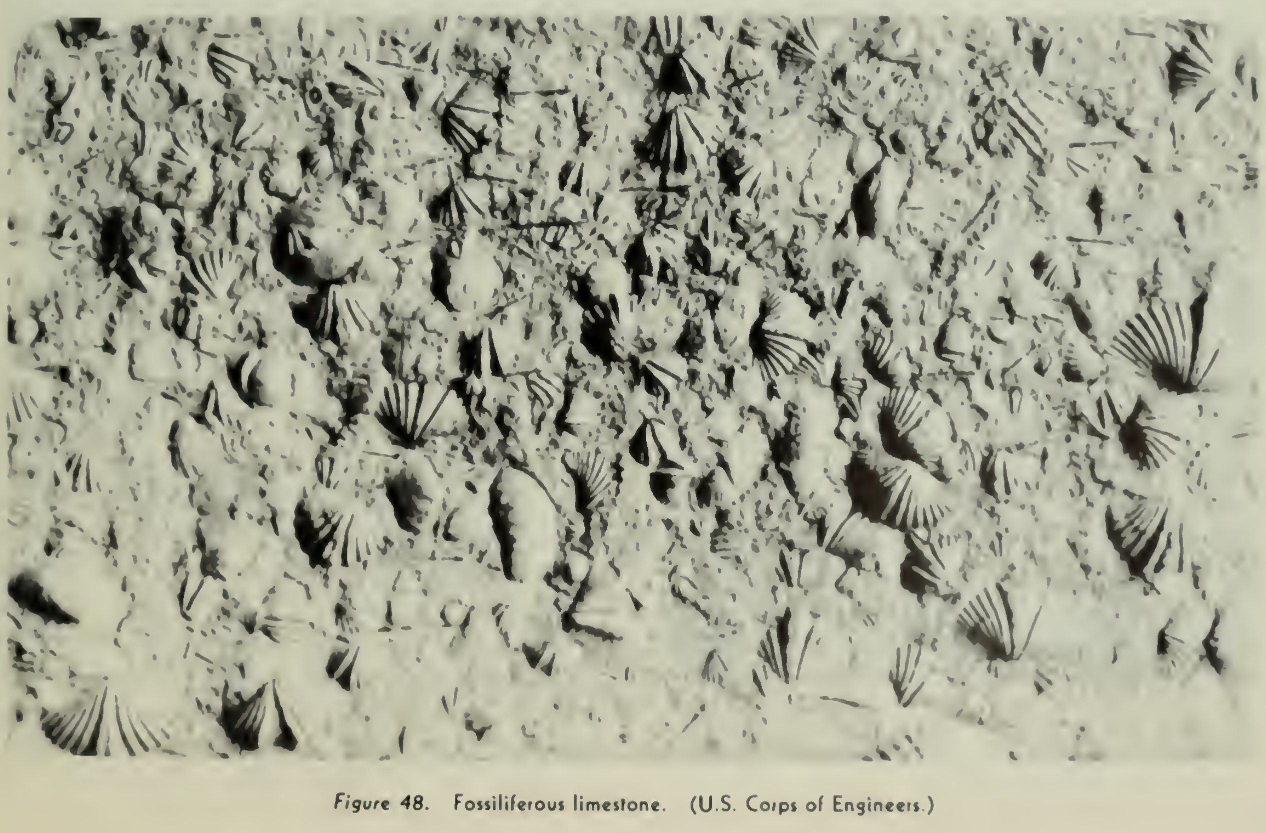


Figure 48. Fossiliferous limestone. (U.S. Corps of Engineers.)

Fluids and gases.—Water is the most important of the liquids and gases involved in metamorphism. Under heat and pressure water becomes a powerful chemical agent. It acts as a solvent, promotes recrystallization, and takes part in the composition of minerals for which it is essential. Water may be reinforced locally by carbon dioxide and fluids issuing from igneous magmas.

(b) *Classification of Common Metamorphic Rocks.*—Metamorphic rocks, on the basis of their primary structure, are readily divided into two groups: foliates and nonfoliates. The foliated metamorphic rocks display a pronounced primary banded or layered structure as a result of the differential pressure to which they have been subjected (fig. 49). The nonfoliated or massive metamorphic rocks do not exhibit primary structural features. Metamorphism has apparently been limited to the process of recrystallization without the action of differential pressure. These structural differences are used as the basis for the simplified classification of the common metamorphic rocks listed in table 9.

Gneiss is characterized by rough, relatively coarse banding or foliation. The bands, often of unlike minerals, commonly appear as alternating light and dark lens-shaped masses in the body of the rock. The common minerals or mineral groups present in gneisses are quartz and the feldspar, mica, amphibole, and pyroxene mineral groups. The specific name assigned is determined by the conspicuous mineral in the rock. For example, gneiss with a predominance of the mineral hornblende would be called hornblende gneiss.

Figure 49. Foliation in metamorphosed sedimentary rocks. (U.S. Corps of Engineers.)

TABLE 9.—*Common metamorphic rocks*

FOLIATED

Texture	Rock	Characteristics
Coarse-grained	Gneiss	Streaked or banded; imperfectly foliated.
Medium-grained	Schist	Well foliated; splits easily; generally rich in mica.
Fine-grained	Slate	Splits readily into smooth sheets.

NONFOLIATED OR MASSIVE

Mineral content	Rock	Characteristics
Chiefly quartz	Quartzite	Hard and brittle.
Chiefly calcite (or dolomite).	Marble	Hard and brittle.
Chiefly hydrous magnesium silicate.	Some types of serpentine.	Fairly soft; green.

Schist is more homogeneous in appearance and composition than is gneiss. The foliae are much thinner, generally more uniform in thickness, finer textured, and often folded or "crinkled" to a much greater degree than the bands of most gneisses. The minerals are, in general, the same as for gneiss, except that talc, chlorites, serpentine, and graphite may be dominant in some schists. As in gneiss, the specific name of a schist is determined by the predominant mineral present.

Slate is very fine grained and homogeneous. Foliation is developed to a very great degree, thus enabling the slate to split into thin sheets with relatively smooth surfaces. The predominant minerals in slate are quartz, mica, chlorite, and sometimes graphite.

Quartzite is a metamorphic rock derived from sandstone by the recrystallization of or cementation by quartz. Quartzite formed by recrystallization bears little resemblance to the parent rock. That formed by cementation exhibits the same physical appearance of the rock from which it was derived. Differentiation between the quartzite resulting from cementation and the sedimentary rock from which it was derived,

therefore, lies in the degree of cementation. The degree of cementation is reflected in the appearance of a fresh fracture. In quartzite, the cementing material is as hard as the sand grains and, therefore, the fracture surfaces are smooth. In sandstone the cementing material is weaker than the sand or silt particles, and therefore, the fracture surfaces are rough. The rough surface is produced by the sand or silt grains which stand above the fracture surface of the weaker cementing material.

Marble is a massive metamorphic rock and has essentially the same mineral content as the lime-type sedimentary rocks from which it is derived.

94. *Engineering Properties of Rocks.* When exposed to the weather long enough, the hardest and most durable rock is ultimately broken down by physical and chemical agents into loose, unconsolidated material or soil. Hence, the physical properties of a rock depend to a large extent on the degree of weathering. For fresh, unweathered rock, the physical properties are affected by the properties of the constituent minerals; the degree to which the mineral grains are bound together; the size and arrangement of the grains which produce such structures as banding and foliation; and the degree of fracturing, jointing, and bedding of the rock mass. The physical properties are the least variable in the igneous rocks, excluding the effects of fracturing. Sedimentary rocks, on the other hand, are so variable that it is difficult to characterize their physical properties which may range between wide limits. Average or typical ranges in properties can be established for sound, unweathered specimens of the common rock types, but in practice each deposit must be evaluated individually. Some of the important rock properties of engineering significance are weight, porosity, strength and hardness, and durability and toughness.

The heaviest rocks are the dark igneous and metamorphic rocks, such as basalt, gabbro, and some schists, which have an average specific gravity of 2.9 to 3.2. The other dense compact rocks, such as granite, slate, marble, and some limestones, have a specific gravity of about 2.5 to 2.8. Lightest are the sedimentary rocks and volcanic rocks such as chalk, tuff, and pumice, which contain many voids. Pumice is generally so light that it floats in water.

In general, the strongest rocks are most dense, and the weakest rocks are most porous. Porosity of granite and similar igneous rocks and of most metamorphic rocks is low, generally less than 1 percent. Basalt is similarly dense, but in certain areas it may contain many small cavities (vesicles) and in other areas it may be extremely vesicular. Porosity of limestone ranges from 0.5 to 15 percent, and in unusual types, such as coquina, up to 25 percent. The porosity of sandstone is typically high, ranging from 5 to 25 percent.

Among the strongest and hardest rocks are quartzites, igneous rocks like granite and basalt, sound gneiss, and some schist. Compressive strengths of 15,000 to 30,000 pounds per square inch or more can be obtained. Some of the hardest, densest sandstones and siliceous limestones approach these strengths. Most limestones, marbles, dolomites, and sandstones, however, are intermediate in strength and hardness with compressive strengths of about 2,500 to 15,000 or 20,000 pounds per square inch. The weakest and softest rocks include tuff, shale, chalk, soft sandstone, salts and gypsum. The softest rocks are easily cut with hand tools. Most limestones and marbles can be sawed. Sandstones, igneous rocks, and the metamorphic rocks composed of quartz and other hard minerals cannot be sawed or cut readily.

The most durable rocks are igneous rocks and massive quartzite and gneiss, but they do not resist fire which causes cracking and spalling. Of these rocks, the fine-grained varieties, such as basalt, are generally tougher and wear better under abrasion than coarse-grained varieties. Foliated and laminated metamorphic rocks, such as schist and slate, are hard but split readily and fall apart under abrasion. In general, limestones and sandstones are moderately tough under abrasion. Limestone and sandstone with limy cement are corroded by water or atmosphere containing acids. Chalk and some tuff are soft and easy to handle in construction but harden on exposure. Shale is weak, tends to soften when wet, and disintegrates rapidly when exposed to weather.

E. SURFACE EXPLORATIONS

95. ***General.***—A relationship between topographic features or landforms and the characteristics of the subsurface soils has been shown repeatedly. Thus, the ability to recognize terrain features on maps, on airphotos, and during field reconnaissance, combined with an elementary understanding of geological processes, can be of great assistance in locating sources of construction materials and in making a general appraisal of foundation conditions.

The mechanisms which develop soil deposits are water action, ice action, and wind action for transported soils; and the mechanical-chemical action of weathering for residual soils. For the transported soils, each type of action tends to produce a group of typical landforms, modified to some extent by the nature of the parent rock. Soils found in similar locations within similar landforms usually have the same physical properties. The engineer responsible for explorations for foundations and construction materials for dams should become familiar with landforms and with the soils associated with them. Such knowledge is of great assistance during the reconnaissance stage of investigations, and it is useful in controlling the extent of investigation for feasibility and specifications designs.

96. ***Fluvial Soils.***—(a) *Definition.*—Soils whose properties are affected predominantly by the action of water to which they have been subjected are designated fluvial soils. Their common characteristic is roundness of individual grains. Frequently, there is considerable sorting action, so that a deposit is likely to be stratified or lensed. Individual strata may be thin or thick, but the material in each stratum will have a small range of grain sizes. The three principal types of fluvial soils, reflecting the water velocity of deposition, are identified as torrential outwash, valley fill, and lake beds.

(b) *Torrential Outwash.*—The typical landforms of this type are alluvial cones and alluvial fans. They vary in size and character from small, steeply sloping deposits of coarse rock fragments to gently sloping plains of fine-grained alluvium several square miles in area. The deposition results from the abrupt flattening of the stream gradient that occurs at the juncture of mountainous terrain and adjacent valleys or plains. Figure 50 is an airphoto and topographic map of an alluvial fan. The coarser material is deposited first; hence, it is found on the steeper slopes at the head of the fan, while the finer material is carried to the outer edges. In arid climates where mechanical rather than chemical weathering predominates, the cones and fans are composed largely of rock fragments, gravel, sand, and silt. In humid climates where the landforms have less steep slopes, the material is expected to contain much more sand, silt, and clay.

Sands and gravels from these deposits are generally subrounded to subangular in shape, reflecting movement over relatively short distances, and the deposits have only poorly developed stratification. The torrential outwash deposits are likely sources of sand and gravel for pervious and semipervious embankment materials and for concrete aggregate. The presence of boulders is likely to limit their usefulness for some types of fill materials. The soils are typically skip-graded, resulting in a GP or SP classification. Because this type of deposit is consolidated only by its own weight, settlement should be anticipated when it is used as a foundation for a structure. Normally, torrential outwash deposits do not provide satisfactory abutments for dams. If it is necessary to locate a dam in the vicinity of such a deposit, the dam should be placed along the upstream edge of the fan.

(c) *Valley Fill.*—Valley fill or flood-plain deposits are generally finer, more stratified, and better sorted than are torrential outwash deposits. The degree of variation from the latter depends largely on the volume and on the gradient of the stream. The surface of these stream deposits is nearly flat. The nature of the materials in the deposit can be deduced by the characteristics of the stream. Braided streams usually indicate the presence of silt, sand, and gravel, whereas meandering streams in broad valleys are commonly associated with fine-grained soils (silts and clays).

Flood-plain deposits of sand and gravel are common sources of concrete aggregate and pervious shell materials for dam embankments. The soils in the various strata of river deposits may range from pervious to impervious; hence, the permeability of the resulting material sometimes can be influenced appreciably by the choice of depth of

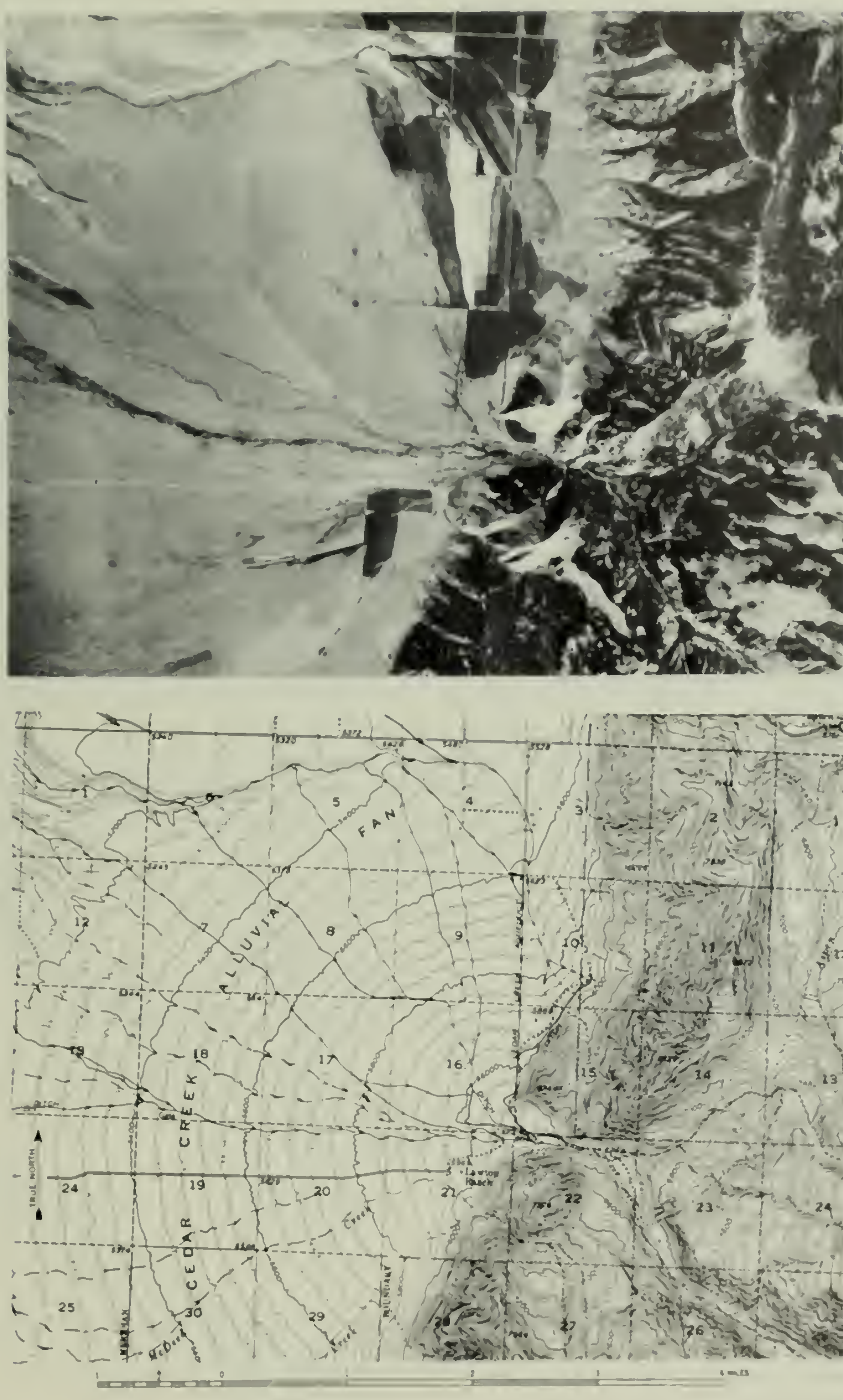


Figure 50. Aerial view and topography of an alluvial fan, a potential source of sand and gravel

cut. Presence of a high water table is a major difficulty in the use of these deposits, especially as a source of impervious material. Also, removal of materials from the reservoir floor just upstream from a damsite may be undesirable when a positive foundation cutoff is not feasible. When considering borrowing from a river deposit downstream from a dam, it should be remembered that such operations may change the tailwater characteristics of the stream channel and that the spillway and outlet works will have to be designed for the modified channel conditions. If tailwater conditions will be affected, borrow operations must produce a predetermined channel and explorations for the specifications stage must accurately define conditions within the channel.

Stream deposits vary in competency as foundations for dams. Potential difficulties include high water table, variation in soil properties, seepage, consolidation, and possibly low shearing strengths. Although valley-fill deposits are usually acceptable as foundations for low dams, their depths and characteristics must be investigated thoroughly during the feasibility stage. An important type of stream deposit is the terrace. It represents an earlier stage of valley development into which the river subsequently has become entrenched. Remnants of such deposits are recognized by their flat tops and steep faces, usually persistent over an extended reach of the valley. Examination of the eroded faces facilitates classification and description of the deposits, and the extent of the drainage network on the face is helpful in determining relative permeability. Free-draining material has almost no lateral erosion channels, whereas impervious clays are finely gullied laterally. Terrace sands and gravels were laid down in the geologic past. These terraces are found along streams throughout the United States and are prevalent in the glaciated regions of the northern sections. Sands and gravels from terrace deposits usually occur in layers and are well graded. They provide excellent sources of construction materials. Figure 51 is an airphoto and topographic map showing river alluvium and terrace deposits.

(d) *Lake Beds.*—Lake sediments, or lacustrine deposits, are the result of sedimentation in still water. Except near the edges of the deposits where alluvial influences are important, the materials are very likely to be fine-grained silt and clay. The stratification is frequently so fine that the materials appear to be massive in structure. Lacustrine deposits are recognizable by their flat surfaces surrounded by high ground. The materials they contain are likely to be impervious, compressible, and low in shearing strength. Their principal use is for impervious cores of earthfill dams. Moisture control in these soils is usually a problem, since the water content is very difficult to change. Lake sediments almost certainly will provide very poor foundations for structures. Their use as foundations for dams is beyond the scope of this text, and should not be attempted without special field and laboratory testing and the advice of a specialist.

97. *Glacial Deposits.*—(a) *Definition.*—The results of the advances and retreats of the great North American continental ice sheets during glacial times are represented by recognizable landforms which are important sources of construction materials and which may be encountered in the foundations of dams. Smaller scale evidences of ice action are found in high mountain valleys of the Rocky Mountains and the Sierra Nevadas; in some instances the glaciers still exist. Glacial deposits (glacial drift) are generally heterogeneous and erratic in nature; hence, they are difficult to explore economically. They contain a wide range of particle sizes, from clay or silt up to huge boulders; and the particle shapes of the coarse grains are typically subrounded or subangular, sometimes with flat faces. Deposits of the glacier proper can be distinguished from deposits formed by the glacier melt water.

(b) *Morainal Deposits.*—Glacial till is that part of the glacial drift deposited directly from the ice with little or no transportation by water. It consists of a heterogeneous mixture of boulders, cobbles, gravel, and sand in an impervious matrix of generally nonplastic fines. Gradation, type of rock minerals, and degree of weathering found in till vary considerably, depending on the type of rocks in the path of the ice and the degree of leaching and chemical weathering. Glacial tills usually produce impervious materials with satisfactory shearing strength, but removal of boulders will be necessary in order for the soil to be compacted satisfactorily. Fairly high inplace density caused by the weight of the ice makes morainal deposits satisfactory for foundation of low dams. Typical land forms containing till are the *ground*

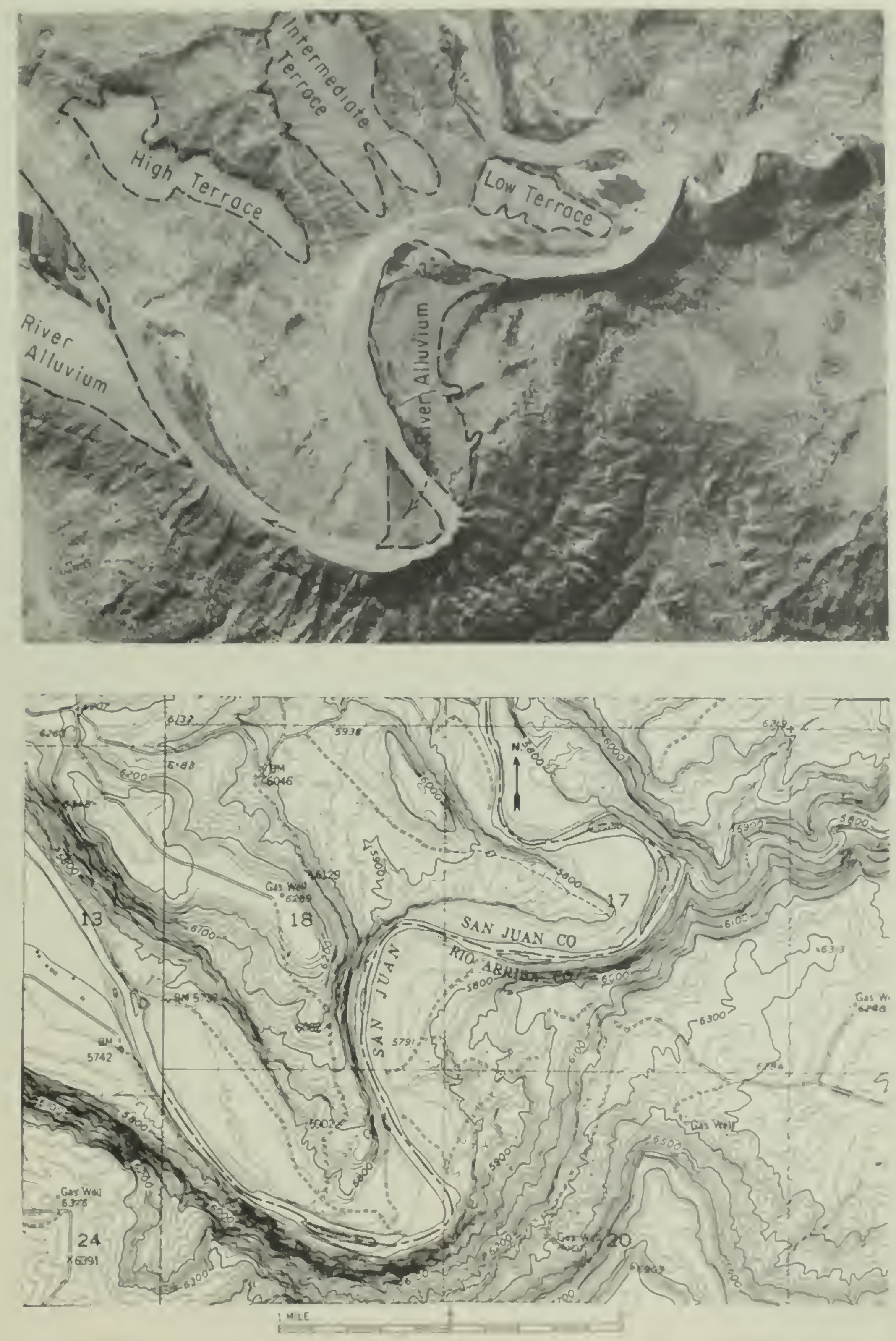


Figure 51. Aerial view and topography of stream deposit showing river alluvium and three levels of gravel terraces. (Photo by Geological Survey.)

moraine or till plain which has a flat to slightly undulating surface; the *end* (*or terminal*) *moraine,* a ridge at right angles to the direction of ice movement, which often curves so that its center is farther downstream than its ends; and *lateral* or *medial-moraines* which occur as ridges parallel to the direction of ice movement. Low, cigar-shaped hills occurring on a ground moraine, with their long axis parallel to the direction of ice movement are called drumlins. They commonly contain unstratified fine-grained soils. Figure 52 shows a typical terminal moraine in an airphoto and topographic map.

(c) *Glacial Outwash.*—Deposits from the glacial melt water are of several types. *Glacial outwash plains* of continental glaciation and their alpine glaciation counterparts, the *valley trains,* commonly contain poorly stratified silt, sand, and gravel similar to the alluvial fans of torrential outwash which they resemble in mode of formation. *Eskers* are prominent winding ridges of sand and gravel which are the remnants of the beds of glacial streams that flowed under the ice. They generally parallel the direction of ice movement, have an irregular crestline, are characterized by steep flanks (about 30°), and are 20 to 100 feet high. Eskers usually contain clean sand and gravel with some boulders and silty strata which are in irregular, poor to fair stratification. They are excellent sources of pervious materials and concrete aggregate. *Kames* are low, dome-shaped partially stratified deposits of silt, sand, and gravel formed by hidden glacial streams. They are round to elliptical in plan with the long axis generally at right angles to the direction of ice movement. Their slopes, contents, and uses are similar to eskers. Glacial lake deposits formed in temporary lakes during the ice age are generally similar in character and in engineering uses to fluvial lacustrine deposits. However, they are normally more coarsely stratified (varved) than are the recent lake deposits, and they may contain fine sand.

98. *Aeolian Deposits.*—Soils deposited by the wind are known as Aeolian deposits. The two principal classes that are readily identifiable are dune sands and loess. Dune sand deposits are recognizable as low elongated or crescent-shaped hills, with a flat slope windward and a steep slope leeward of the prevailing winds. Usually, these deposits have very little vegetative cover. The material is very rich in quartz and its characteristics are limited range of grain size, usually in the fine or medium range sand; no cohesive strength; moderately high permeability; and moderate compressibility. They generally fall in the SP group of the Unified Soil Classification System.

Loessial deposits of windblown dust cover extensive areas in the plains regions of the temperate zone. They have a remarkable ability for standing in vertical walls. Figure 53 shows typical loessial topography by map and airphoto. Loess consists mainly of angular particles of silt or fine sand, with a small amount of clay that binds the soil grains together. In its natural state true loess has a characteristic structure formed by remnants of small vertical root holes that makes it moderately pervious in the vertical direction. Although of low density, the naturally dry loessial soils have a fairly high strength because of the clay binder. This, however, may be lost readily upon wetting, and the structure may collapse. When remolded, loessial soils are impervious, moderately compressible, and of low cohesive strength. They usually fall in the ML group or the boundary ML–CL group of the Unified Soil Classification System. Figure 54 shows a 90-foot almost vertical cut in loess.

Aeolian deposits are normally regarded with suspicion, especially as foundations for dams. Such deposits should be avoided if it is practicable to do so; however, they can be used when properly explored and evaluated. Information on the in-place density of aeolian soils is of vital importance in planning their usefulness for foundations of structures.

99. *Residual Soils.*—As weathering action on rock progresses, the rock fragments are gradually reduced in size until the total material assumes all the characteristics of soil. It is impracticable to define clearly the dividing line between rock and residual soil, but for engineering purposes it may be considered soil if the material can be removed by commonly accepted excavating methods.

A distinguishing feature of many residual soils is that the individual grains are angular but soft. Handling of the material during construction reduces appreciably the grain size, which makes it difficult to predict its performance by laboratory tests. Appreciable settlement and change of material characteristics after utilization also are detrimental factors. As a consequence, such residual

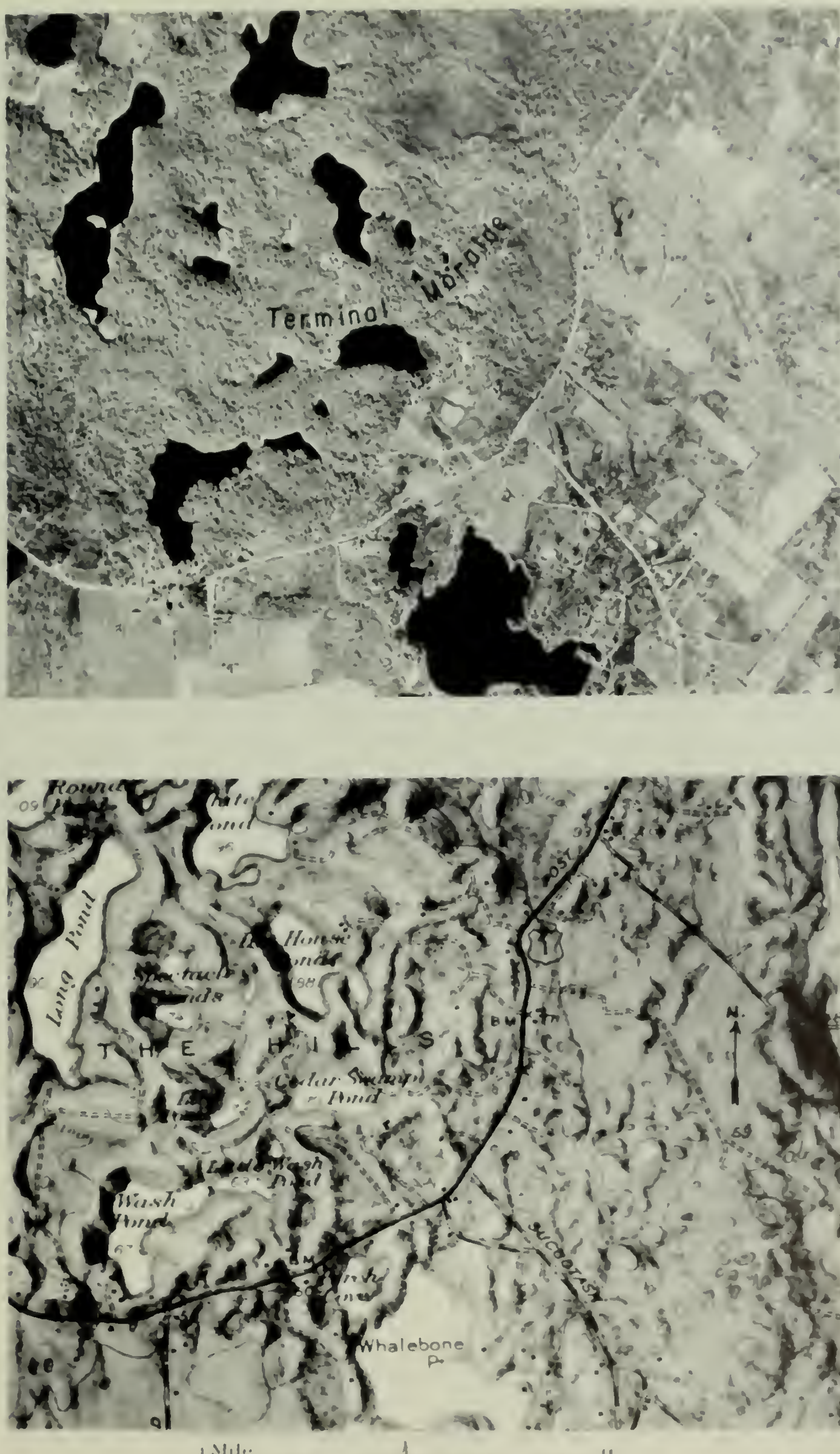


Figure 52. Aerial view and topography of terminal moraine of continental glaciation. (Photo by U.S. Commodity Stabilization Service.)

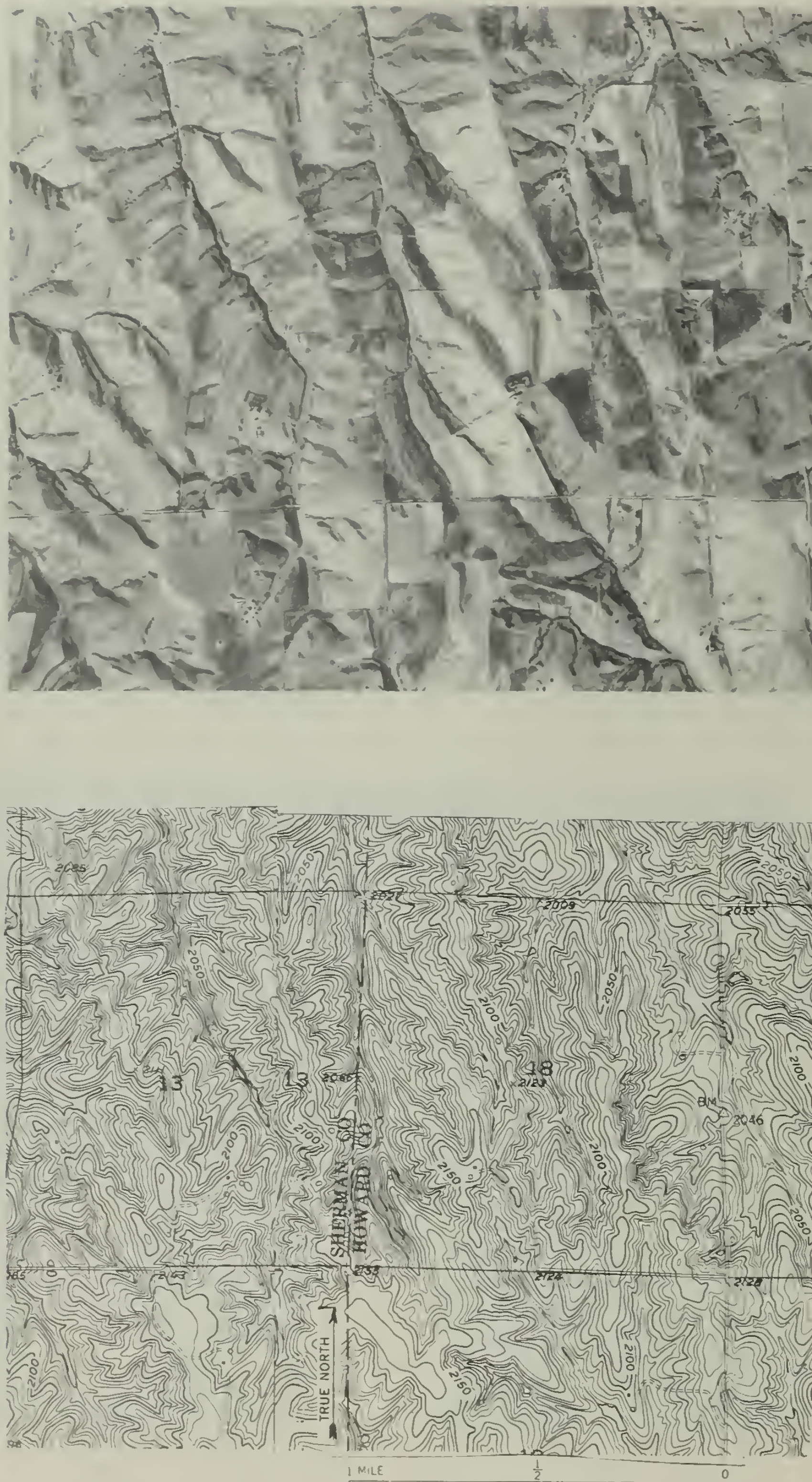


Figure 53. Aerial view and topography of loess identified by smooth silt ridges; usually parallel, right-angle drainage patterns; and steep-sided, flat-bottomed gullies and streams. (Photo by U.S. Commodity Stabilization Service.)

Figure 54. A 90-foot nearly vertical cut in loess formation—Nebraska.

soils are best avoided if other types can be readily secured.

It is difficult to recognize and appraise residual soils on the basis of topographic forms. Their occurrence is quite general wherever none of the other types of deposits, with their characteristic shapes, are recognizable and where the material is not clearly rock in place. Talus (fig. 25) and landslides are easily recognizable forms of residual materials moved by gravity. Also, erosional features may be helpful in evaluating a residual deposit. Since the type of parent rock has a very pronounced influence on the character of the residual soil, the rock type should always be determined in assembling data for the appraisal of a residual deposit. The soils usually provide satisfactory foundations for small structures if the parent rock itself is satisfactory. The degree to which alteration has progressed largely governs the strength characteristics.

F. SUBSURFACE EXPLORATORY METHODS

100. ***Test Pits, Trenches, and Tunnels.*** (a) *General.* Open test pits, trenches, and drifts afford the most complete information of the ground penetrated and also may permit examination of the surface of foundation bedrock. When the depth of overburden and ground-water conditions permit their economical use, these methods are recommended for foundation exploration in lieu of relying solely on borings. In prospecting for embankment materials or concrete aggregate containing

cobbles and boulders, open pits and trenches may be the only feasible means of obtaining the required information.

(b) *Test Pits.*—Test pitting is an effective means of exploring and sampling earth foundations and construction materials. Their use facilitates inspection, sampling, and making density tests. The depth of a test pit is determined by investigational requirements but is usually limited to a few feet below water table. The minimum recommended cross section for a hand-dug pit is 3 by 5 feet. Dragline, backhoe, clamshell, and bulldozer pits are usually more economical than hand-dug pits for comparatively shallow materials explorations, but are not practicable where a depth of more than about 15 feet is desired. Where the soil is hard, pneumatic paving breakers or spades operated by small trailer-mounted air compressors will facilitate progress in hand-dug pits. The use of explosives to break up large boulders is a common practice.

In hand-dug pits the materials are removed from the hole by buckets operated from a hoist or windlass which should be equipped with a ratchet device for safety. During excavation, the bottom of the hole should be kept fairly level and of full size so that each lift may represent the corresponding portion of the deposit in quantity and quality.

At the surface the excavated material should be placed in an orderly manner around the pit, and marked stakes should be driven to indicate depth of pit from which the material came in order to facilitate logging and sampling. All hand-dug pits should be cribbed. A convenient method of cribbing, with 3- by 6-inch lumber, is shown in figure 55. In loose materials it is advisable to keep the space beteeen the pit walls and the cribbing at a minimum, also to pack the space with hay or excelsior and to keep the bottom of the cribbing close to the bottom of the pit.

Deep test pits should be ventilated to prevent accumulation of dead air. For this purpose connected lengths of stovepipe, starting slightly above the floor and extending about 3 feet above the mouth of the pit, have been found satisfactory. Test pits left open for inspection should be provided with covers or barricades for safety.

When water is encountered in the pit, a pumping system is required for further progress. Small, portable, gas-powered, self-priming, centrifugal pumps can be used. It is desirable that

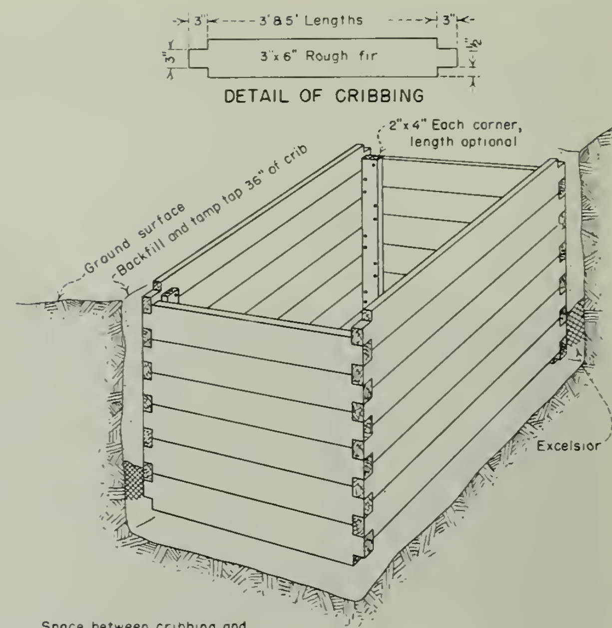


Figure 55. **Test pit cribbing.**

the suction hose be one-half inch larger in diameter than the discharge opening of the pump and not more than 15 feet long. This requires resetting the pump in the pit (on a frame attached to the cribbing) at intervals of about 12 feet. When the gas engine is in the pit, it is necessary to pipe the exhaust well away from the pit. Unwatering test pits is usually expensive and often is unwarranted. Figures 56 and 57 show different types of test pits.

(c) *Trenches.*—Test trenches are used to provide a continuous exposure of the ground along a given line or section. In general, they serve the same purpose as the open test pits but have the added advantage of disclosing the continuity or character of particular strata. They are best suited for shallow exploration (10 to 15 feet) on moderately steep slopes, but they have been used advantageously on fairly flat ground.

The fieldwork consists of excavating an open trench from the top to the bottom of the slope to reach representative undisturbed material. Either a single slot trench down the face of the slope or a series of short trenches spaced at appropriate intervals along the slope can be excavated. Depending on the extent of the investigation required, either pick and shovel methods, or bull-

Figure 56. Excavation of hand-dug test pit in borrow area. SB-69-R2.

Figure 57. Equipment-excavated test pit showing elevation of ground, water surface, bedrock, and location of samples. LM–166.

dozers, ditching machines, or draglines can be used. A trenching layout suitable for materials investigations is shown in figure 58. Figures 59 and 60 are photographs of bulldozer trenches. Safety precautions should be used in deep trenches to prevent accidents caused by caving ground.

The profile exposed by these trenches may represent the entire depth of significant strata in an abutment of a dam; however, their shallow depth may limit exploration to the upper weathered zone of foundations. Trenching permits visual inspection of the soil strata which facilitates logging of the profile and selection of samples. It also aids in obtaining large undisturbed samples or large disturbed individual or composite samples. Trenches in sloping ground have the advantage of being self-draining.

(d) *Tunnels.*—Tunnels or drifts have been used to explore areas beneath steep slopes or back of clifflike faces. For such purposes the exploratory tunnel or drift is usually roughly rectangular in shape and approximately 5 feet wide and 7 feet high. The placing of lagging, when required for side and roof supports, should follow excavation as closely as practicable. Excavation of exploratory tunnels may be a slow, expensive process; consequently, this type of investigation should be utilized only when no other method will supply the required information. Logging and sampling of exploratory tunnels should proceed concurrently with excavation operations if possible.

101. *Auger Borings.*—Auger borings often provide the simplest method of soil investigation and sampling. They may be used for any purpose where disturbed samples are satisfactory and are valuable in advancing holes to depths at which undisturbed sampling by thin-walled tubes is required. Depths of auger investigations are, however, limited by the ground-water table and by

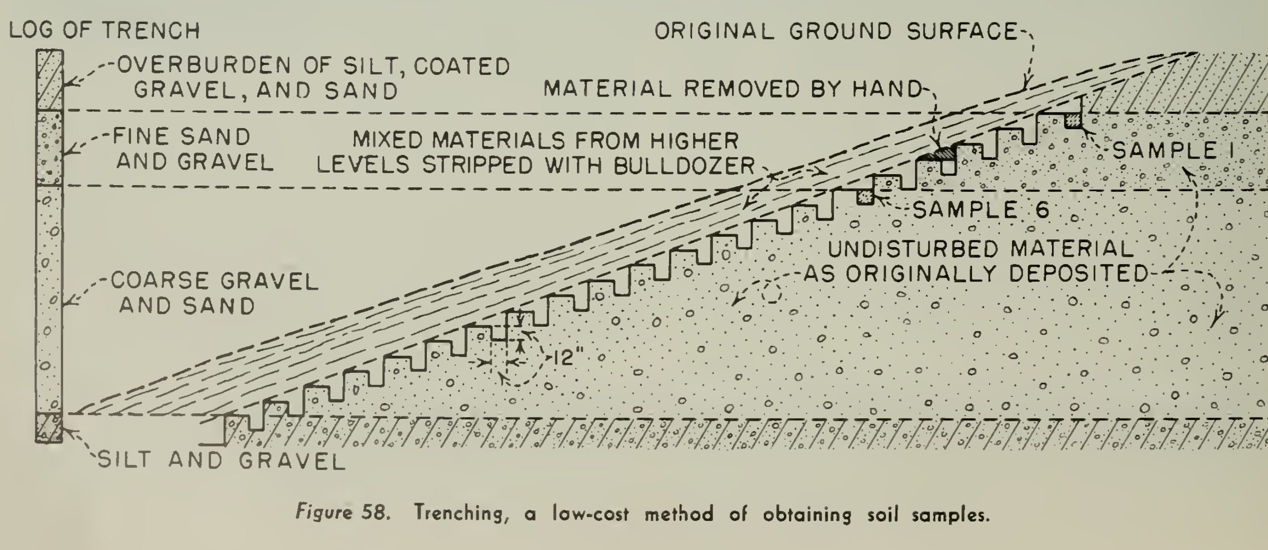


Figure 58. Trenching, a low-cost method of obtaining soil samples.

Figure 59. Tractor with bulldozer excavating a test trench. SB–61–R2.

the amount and maximum size of gravel, cobbles, and boulders as compared with the size of equipment used. Hand-operated post-hole augers 4 to 12 inches in diameter can be used for exploration up to about 20 feet deep (figs. 61 and 62). However, with the aid of a tripod, holes up to 80 feet deep have been excavated successfully and economically (fig. 63). Machine-driven augers are of three types: Helical augers 3 to 16 inches in

Figure 60. Bulldozer-excavated trench in steep abutment area.

Figure 61. Exploring a borrow area with a hand auger.

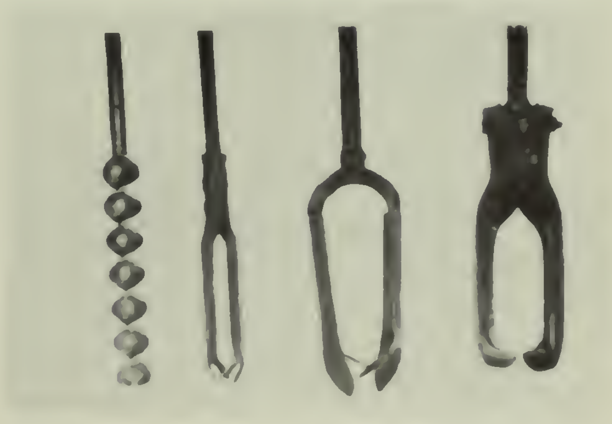


Figure 62. Types of hand augers (2-inch helical; 2- and 6-inch Iwan; and 6-inch Fenn (adjustable)).

diameter; disk augers up to 42 inches in diameter; and bucket augers up to 48 inches in diameter. Figure 64 shows helical augers, including a helical pipe casing (shown in the machine) which is closed on the bottom during placement but is opened by means of a smaller inserted pipe to permit sampling or testing of lower strata through it. Figure 65 shows a disk auger, and figure 66 shows a bucket auger.

Figure 63. **Exploration of borrow area with project-designed demountable tripod used to facilitate hand augering of 4-inch holes to a depth of 55 feet. The guide ring is set at 30 feet. 404–1006B.**

An auger boring is made by turning the auger the desired distance into the soil, withdrawing it, and removing the soil for examination and sampling. The auger is inserted into the hole again, and the process is repeated. Pipe casing is required in unstable soil in which the borehole fails to stay open, and especially where the boring is extended below the ground-water level. The inside diameter of the casing must be slightly larger than the diameter of the auger used. Unless the helical casing shown in figure 64 is used, the casing is driven to a depth not greater than the top of the next sample and is cleaned out by means of the auger. The auger can then be inserted into the borehole and turned below the bottom of the casing to obtain the sample.

The soil auger can be used both for boring the hole and for bringing up disturbed samples of the soil encountered. It operates best in somewhat loose, moderately cohesive, moist soil. Holes are usually bored without the addition of water; but

Figure 64. **Drilling machine, with helical augers in foreground and helical casing in the machine.**

Figure 65. **Disk auger used to explore borrow areas of fine-grained soils.**

Figure 66. **Bucket auger used in exploration of a borrow area containing gravel particles.**

in hard, dry soils or in cohesionless sands the introduction of a small amount of water into the hole will aid the drilling and sample extraction. Rock fragments larger than about one-tenth of the diameter of the hole cannot be successfully removed by normal augering methods. Large-sized holes permit examination of the soils in place, and therefore, are preferred for foundation investigations.

102. ***Rotary Drilling.***—One of the most important tools for subsurface exploration of dams is the diamond drill: A rotary drill with a core barrel, a diamond bit, and a hydraulic or screw feed. Core sizes are designated as NX, BX, AX, and EX, which produce cores $2\frac{1}{8}$, $1\frac{5}{8}$, $1\frac{1}{8}$, and $\frac{7}{8}$ inches in diameter, respectively. The diamond drill may actually be operated with a variety of bits, depending on the hardness of the material penetrated. Rotary drill equipment is manufactured in a wide variety of forms, which vary from highly flexible to extremely specialized equipment, from lightweight and highly mobile equipment to heavy stationary plants, and in size of hole and core range from less than 1 inch to 3 feet or more. They are capable of drilling to depths far below those required for small dams.

Essential accessories for a drill rig are a cathead winch and derrick for driving casing and for hoisting and lowering the drill rods; a pump for circulating water to the bit and for flushing and water testing the hole; a water meter; and the necessary driving weights, bits, drill rods, and core barrels. Cased holes are usually required except when drilling through solid rock or stiff, cohesive soils. A short collar pipe about 5 feet long is commonly used at the top of the hole. The use of drilling mud, including hole stabilizer compounds, sometimes avoids the need of casing in overburden soil, but the foundation cannot be effectively water tested when mud is used.

At least two cast-iron weights should be available, a 140-pound weight for standard penetration tests and a 250- to 400-pound weight for driving and removing casing pipe. The weights are raised by pulling tight on an attached rope threaded through a sheave at the top of the derrick and wound three or four times on the revolving cathead winch. Sudden loosening of the rope permits the weight to drop on the driving head attached to the casing. Various types of chopping bits are used to facilitate the driving of casing through soils containing cobbles and boulders. Large boulders must be either blasted with explosives or drilled with a plugged diamond bit. The casing is raised several feet prior to blasting. Figure 67 shows a diamond drill rig with derrick.

The accuracy and dependability of the records furnished by diamond drilling depend largely on the size of the core in relation to the kind of material drilled, the percentage of core recovery, the behavior during drilling, and the experience of the drill crew. Since a rock that will core well in an NX hole may break up badly in an EX hole, it is important to use the largest practicable size diameter core. Recovery of core is much more important than making rapid progress in drilling the hole. Portions of a core that are lost will probably represent shattered or soft, incompetent rock, whereas the recovered portions represent the best rock, from which an overoptimistic evaluation of the foundation likely will be made. A reasonably large percentage of core recovery, on the other hand, will provide a more or less continuous

Figure 67. **Diamond-drill rig used in exploration of a foundation for a dam.**

section of the materials passed through. The cores provide information on the character and composition of the different formations, with evidence of the spacing and tightness of joints, seams, fissures, and other structural details. When drilling in soft materials, the water circulation must be reduced or stopped entirely, and the core recovered "dry," even though a marked delay in operations may result.

Core barrels are described in section 107. The barrel is fitted with a drill bit and is lowered into the hole by the hollow drill rod. Circulation of the wash water should be started before the core barrel reaches the bottom of the hole to prevent cuttings or sludge from entering the core barrel at the start of coring. The optimum rotation speed of drilling varies with the type of bit used, the diameter of core barrel, and the kind of rock to be cored. Excessive rotation speed will result in chattering and rapid wear of the bit and will break the core. Low speed results in less wear and tear on the bit and better cores but lower rates of progress. Approximate ranges of rotation speeds used in medium-hard rock are 300 to 1,500 revolutions per minute for diamond bits and 100 to 500 revolutions per minute for metal bits.

The rate at which the coring bit is advanced depends on the amount of pressure applied downward on the bit as well as on its speed of rotation. This pressure must be carefully adjusted by the driller; excessive pressures will cause the bit to plug and may shear the core from its base. The bit pressure is controlled by a hydraulic or a screw feed on the drilling machine. The weight of the column of drill rod will seldom be in excess of the optimum bit pressure for the coring of medium and hard rock, and it is frequently necessary to supply additional downward pressure.

Since the hole left in the rock is clean and the seams and fissures are not sealed off by the action of the drill, opportunity is afforded for making percolation tests to indicate the permeability of the strata and to determine probable leakage through open joints or fissures in the rock. Gravity or pressure tests are made as described in section 113. Large water losses or inflows into the

holes during drilling are recorded as indicating, respectively, the presence of large openings in the formation or the existence of an underground flow. Completed holes should be capped to preserve them for use in ground-water level observations or as grout holes, or for reentry if it is later found desirable to deepen the hole. Casing is, of course, required for those sections of the hole in loose material or unconsolidated ground.

Although the rotary drill is designed primarily for penetrating rock rather than soil, sample barrels and cutting bits have been developed which are satisfactory in many kinds of soil deposits. Double-tube core-barrel samplers of the Denison type are capable of obtaining undisturbed samples of sands, silts, and clays for laboratory testing [6]. However, for foundations of small dams within the scope of this text, borings for the standard penetration tests described in the next section should preclude the need for Denison-type samples.

103. *Standard Penetration Borings.* This is a procedure for making soil borings with a split-barrel sampler in order to obtain representative, moderately disturbed samples of soil for identification purposes and to obtain a record of the resistance of the soil to penetration of the sampler. The equipment used should provide a reasonably clean hole at least 2¼ inches in diameter before insertion of the sampler. To assure that the material to be tested is undisturbed, use of bottom-discharge fishtail bits or jetting through an open pipe to clean out the hole should not be permitted. Side-discharge fishtail bits, however, are satisfactory. Either a rotary drill or an earth auger can be used to advance and clean the hole. Casing is used when drilling in sand, soft clay, or other material that will not allow the hole to stay open.

The dimensions of the sampler are shown in figure 68, and figure 69 shows the sampler disassembled. The driving shoe is of hardened steel which must be replaced or repaired when it becomes dented or distorted. Split-barrel samplers other than 2 inches in outside diameter can be used; however, all penetration records made with those samplers should be conspicuously marked as to the size of sampler used. The significance of the penetration records is discussed in section 129. A drive weight assembly consisting of a 140-pound weight, with a 30-inch free fall, is used. A heavier hammer is permitted for driving casing.

Penetration tests should be made continuously in exploring foundations for dams, except where the resistance of the soil is too great. Any loss of circulation in drilling fluid during advancing of holes should be noted and recorded on the log. When casing is used, it should not proceed in

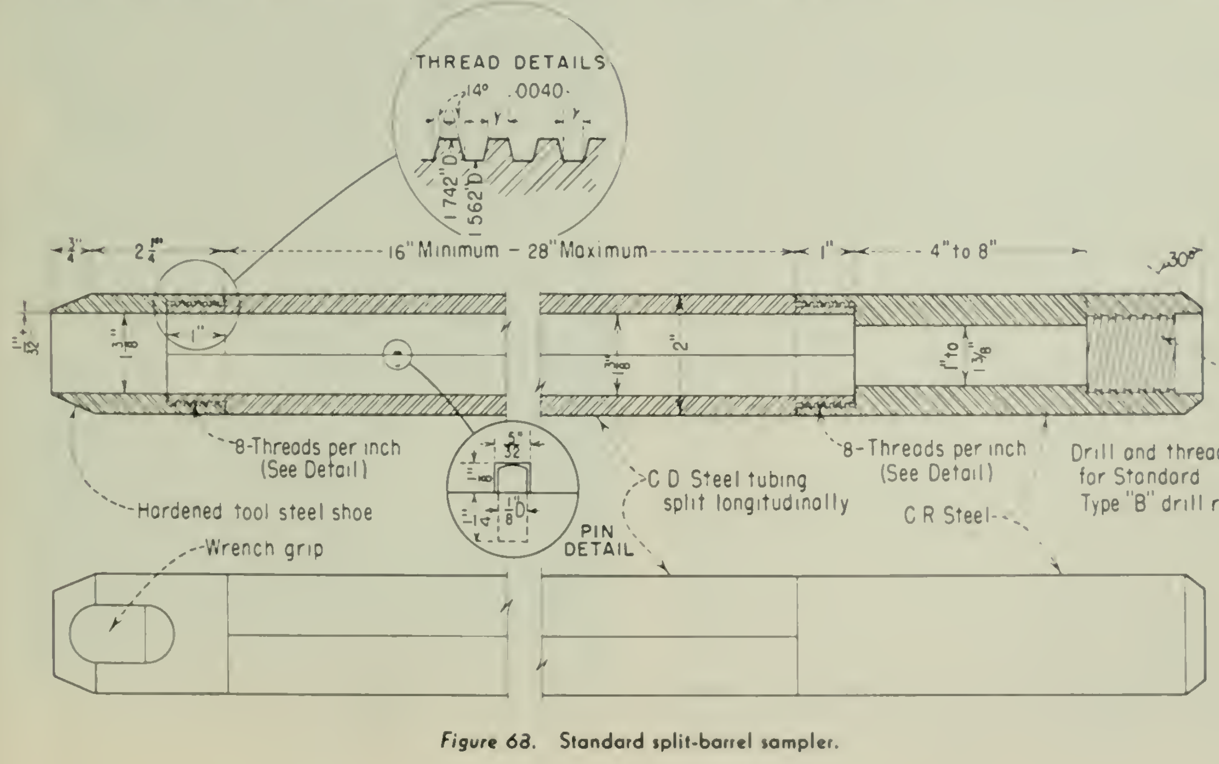


Figure 68. Standard split-barrel sampler.

Figure 69. **Split-barrel sampler disassembled.**

advance of the sampling operation. With the split-barrel sampler resting on the bottom of the hole and the water level in the boring at the ground-water level or above, the sampler is driven through undisturbed soil for about 6 inches; then the test is begun by driving the sampler 12 inches, or to refusal, by dropping the 140-pound hammer 30 inches. (See figs. 70 and 71.) The number of blows required to effect the 12 inches of the penetration test are recorded. If 50 blows result in less than 1-foot penetration, the test is stopped and the data recorded. The sampler is then raised to the surface and opened.

Prior to reinsertion of the sampler, the hole should be cleaned out by an auger or by a rotary drill down to the level of the bottom of the previous test. The test is then repeated. In this manner an almost continuous record of the material encountered is obtained by sampling from the split barrel, and penetration resistance values are available for alternate 1-foot increments of depth of hole. Information on sampling and logging of standard penetration holes is given in parts G and H of this chapter, respectively.

104. ***Geophysical Methods.***—Seismic surveys have been used successfully to determine approximate depths to bedrock in order to select the best of several damsites in advance of drilling and to locate buried channels. Electric resistivity surveys have been used to determine approximate depths of weathering of bedrock and to find the extent of buried gravel deposits.

Both methods require special equipment and experienced operators, and the data obtained must be analyzed and interpreted by trained specialists with appropriate reference to the geologic character of the area. Moreover, geophysical methods require correlation with borings to aid in the interpretation of the data they obtain, and their use is warranted only where they can substantially reduce the number of borings. Since only a relatively small number of borings ordinarily is required for small dams, geophysical methods of exploration are usually not appropriate for these projects.

G. SAMPLING

105. *General.* The purposes and uses of samples are numerous. They are required to identify and classify soil or rock properly. They are essential for field density and moisture determinations and for laboratory tests on earth materials, concrete aggregate, and riprap. To a large degree, samples determine the results of explorations for foundations and construction materials for dams. Obviously, erroneous conclusions will be drawn if the samples are not truly representative of the explorations.

Samples are broadly classified as disturbed and relatively undisturbed. Disturbed samples are those in which no effort is made to maintain the soil structure. Such samples may be secured for general observation and inspection, soil classification, moisture determination, or to obtain compaction characteristics. Relatively undisturbed samples vary from moderately disturbed split-barrel samples to almost completely undisturbed hand-cut samples. The importance of obtaining representative samples cannot be overemphasized. This requires considerable care because of the variations in natural deposits of earth materials. Representative samples are comparatively easy to secure in trenches, test pits, and cut banks, because the various undisturbed strata can be inspected visually. Boreholes, however, do not permit a visual inspection of the profile, and it is more difficult to secure representative samples by use of boreholes.

Figure 70. **Making a standard penetration test by use of a drill rig.**

Samples may be either individual or composite. An individual sample is a single sample representing one stratum or type of soil. A group of individual samples representing all the strata from a single test pit may be combined in the same ratio as the thicknesses of the strata from which the samples were taken to form a composite sample. Also, a sample may be composited to any desired ratio of the soil types involved. It is usually preferable to obtain individual samples from each stratum of a soil deposit rather than to composite the sample in the field, since the most desirable depth of excavation may be determined by trying several composite samples in the laboratory. Representative samples of stockpiles are essentially composite samples in that parts are taken at random over the pile. If the sample is taken from one location, the selected area should be excavated deep enough to avoid segregated materials.

The sizes of samples required depend on the nature of the laboratory tests which may be required. Table 10 gives suggested sizes of samples and the information required on a sample identification tag. Disturbed samples of 75 pounds or more should be placed in bags or other suitable containers which will prevent loss of the fine fraction of the soil and moisture. Samples of silty and clayey soils proposed for borrow which are obtained for laboratory Atterberg limits tests and Proctor compaction tests should be protected against drying and should be shipped in waterproof bags or other suitable containers to retain the natural moisture so far as possible. Samples of sands and gravels may be shipped in closely woven textile bags and should be air dried before placing in the bag. When the sack samples are shipped by public carrier, they should be double sacked. The protection and preparation for shipping of undisturbed samples is discussed in section

Figure 71. **Making a standard penetration test by use of a tripod unit.**

TABLE 10.—*Identification and sizes of samples*

[As identification on sample tags, give project name, feature, area designation, hole number, and depth of sample]

Purpose of material	Sample size	Remarks
Individual and composite samples of disturbed earth materials for classification and Proctor compaction tests.	Sufficient material, all passing the 3-inch sieve, to yield 75 pounds passing the No. 4 sieve.	Include information relative to the percentage by volume 3 inches to 5 inches and plus 5 inches.
Soil-rock permeability tests	300 pounds passing a 3-inch sieve	Air dried.
Relative density test	150 pounds passing a 3-inch sieve	Air dried.
Moisture samples, inspection samples of soil, soil samples for sulfate determination (reaction with concrete).	Sealed pint or quart jar (full)	Individual inspection samples should represent range of moisture and type of materials.
Concrete aggregate	600 pounds of pit-run sand and gravel. If screened: 200 pounds of sand, 200 pounds of No. 4 to ¾-inch size, and 100 pounds of each of the other sizes produced. 400 pounds of quarry rock proposed for crushed aggregate.	For commercial sources, include data on ownership plant, and service history of concrete made from aggregates.
Riprap	200 pounds representative of quality	Method of excavation used, location of pit and quarry.
Inplace density and water content of fine-grained soils above water table.	8- to 12-inch cubes or cylinders	Sealed in suitable container.

107. It is recommended that those samples not tested should be stored for possible future examination and testing until the dam is completed and has shown satisfactory operating characteristics.

106. *Disturbed Samples.* (a) *Sampling Open Excavations.* An area of the sidewall of the test pit, trench, or open cut should be trimmed to remove all weathered or mixed material. The exposed strata should be examined for changes in gradation, natural water content, plasticity, uniformity, etc., and the representative strata should be selected for sampling.

Either individual or composite samples are obtained by trenching down the vertical face of a pit, trench, or cut bank with a cut of uniform cross section and collecting the soil on a quartering cloth spread below the trench (fig. 72). The minimum

Individual samples are taken from each layer of soil.
Composite samples are taken from two or more layers of soil

Figure 72. Sampling trench.

cross section of the sampling trench should be at least four times the dimensions of the largest gravel size included in the soil. In taking individual samples it is important to be certain that sufficient representative material is obtained from the stratum and to see that extraneous material is not included. For composite samples, a vertical trench is cut through all strata above any desired elevation.

If the material being sampled is a gravelly soil that contains large percentages (approximately 25 percent or more of the total material) of particles 3 inches or larger in size, it is usually advantageous to take representative proportional parts of the entire excavated material, such as every fifth or tenth bucketful, to be saved as a part of the sample rather than to trim the sample from the sidewall of the excavation. When the samples are larger than required for testing, their size may be reduced by quartering. This is done by piling the total sample in a cone on a canvas or tarpaulin, each shovelful going to the center of the cone and being allowed to run down equally in all directions. The cone is then spread out in a circle by walking around the pile and gradually widening the circle with a shovel until a uniform thickness of material has been obtained. The flat pile is then marked into quarters. Two opposite quarters are discarded. The material in the remaining quarters is mixed again by shoveling the material into another conical pile, taking alternate shovelfuls from each of the two quarters. The process of piling, flattening, and rejecting two quarters is continued until the sample is reduced to the desired size.

(b) *Sampling Auger Holes.*—Small auger holes cannot be sampled and logged as accurately as an open trench or a test pit, since the inaccessible auger hole does not permit visual inspection of the total profile and selection of representative strata. Small hand auger (4-inch diameter or less) samples are adequate for soil classification, but do not provide sufficient material for testing purposes. An 8-inch-diameter hand auger, however, will provide samples of sufficient size for testing. As the auger hole is advanced, each amount of soil extracted by the auger should be deposited in individual stockpiles nearby to form an orderly depth sequence of removed material. In preparing an individual sample from an auger hole, first, group the various consecutive piles together to form representative samples of the various strata; and, second, mix all or equal parts from each of the appropriate stockpiles together to form the desired size of sample from each stratum (fig. 73).

Sampling from large-diameter power augers is accomplished by lifting the auger blade at regular intervals, such as after each 9 to 12 inches of penetration, taking a shovelful of soil from the auger blade, and placing in sacks or stockpiling in an orderly depth sequence as the auger hole is advanced (fig. 65). Consecutive similar stockpiles can be combined to form individual samples.

(c) *Sampling Stockpiles.*—When sampling stockpiles or windrows, care must be taken to see that the samples are not selected from segregated areas.

Figure 73. **Auger sampling.**

The amount of segregation in materials depends on the gradation of the material and on the methods and equipment used for stockpiling. Even with good control the outer surfaces and fringes of a stockpile are likely to have some segregation, particularly if the slopes are steep and the material contains an appreciable amount of gravel or coarse sand. Representative samples from stockpiles should be obtained by combining and mixing small samples taken from several small test pits or auger holes distributed over the pile. A windrow of soil is best sampled by taking all the material from a narrow cut transverse to the windrow. Samples from either stockpiles or windrows should be fairly large originally and should be thoroughly mixed before they are quartered down to the desired size for testing purposes.

(d) *Riprap Samples.*—The competency and quality of rock for riprap are judged by physical properties tests, petrographic examination, and service record of the material. Since the riprap requirements include obtaining proper sizes of rock fragments, quality tests made in the laboratory must be supplemented by data obtained by field examination and the results of blasting tests in proposed quarry sites. The importance of obtaining representative samples of each type of material in a proposed riprap source must be emphasized. If there is more than one type of material in a source, separate samples should be obtained representing each material proposed for use. Intervening layers of soil, shale, or other soft rock obviously unsuitable for riprap need not be sampled, but full descriptions of these materials should appear on the drill logs and in a report of the investigation.

Samples should be obtained by blasting down an open face on the sidewall of a test pit, trench, or exposed ledge to obtain unweathered fragments representing each type of material as it will be quarried and used in riprap. Figure 74 is a photograph of a blast test in a riprap source. The selection of exterior weathered rock fragments will not provide a representative sample unless only exterior weathered fragments are to be used as riprap.

Large fields of boulders are sometimes proposed as sources of riprap. The production of riprap from boulder fields is always a costly process, and should be considered only when ledge materials are not available. Field boulders usually do not have the angularity and interlocking properties of quarried riprap. Sampling of boulder sources should include breaking of large boulders with explosives to obtain samples of fragments similar to those likely to result from operations during construction. Talus slopes should be sampled only if the talus itself is proposed for use as riprap. Samples of talus material generally do not represent the material obtainable from the solid rock ledge above the talus slope, because the talus fragments are generally weathered, altered, or case-hardened.

(e) *Concrete Aggregate Samples.*—Disturbed samples of concrete aggregate materials are obtained from test pits, trenches, and cased auger holes. Since the gradation of concrete aggregates is of great importance, portable screening apparatus is sometimes used to determine the gradings of the samples in the field, thus permitting an estimate of the processing operations which will be required. Figure 75 shows exploration for concrete aggregate using a portable screening device. Whenever facilities are available, representative samples of the aggregate should be subjected to laboratory tests to determine the physical and chemical properties of the material. In the absence of facilities for laboratory tests, examination of the aggregate by an experienced petrographer will aid considerably in estimating its physical and chemical soundness. The performance record of an aggregate in concrete when it can be determined, is of great value in appraising

Figure 74. **Blast test in metamorphic rock investigated as a source of riprap.**

the potential source of concrete aggregate and should be employed whenever possible. Laboratory tests on concrete aggregate are discussed in section 116.

107. *Relatively Undisturbed Samples.* (a) *Penetration Samples.*—The standard penetration boring includes the use of the split-barrel sampler shown in figures 68 and 69. The procedure used in the test under ideal conditions permits the securing of an 18-inch-long sample, the lower 12 inches of which represent the material for which the penetration resistance (in blows) is known. Typical samples of soil from the open split-barrel are placed into jars without ramming. The origin of the sample is noted on the jar, and it is stored in suitable containers for shipment. The jars are sealed with wax or a self-sealing top to prevent evaporation of the soil moisture. Complete identification of the sample, including boring number, sample number, depth, penetration record, and length of recovery are placed on the jar. Samples should be protected from freezing and should not be placed in the sun. The samples are useful for visual classification of the soil and for laboratory tests of water content and Atterberg limits, when required.

(*b*) *Hand-cut Samples.*—Undisturbed samples in the form of cubes, cylinders, or irregular chunks can be obtained from strata exposed in the sides or bottoms of open excavations, test pits, trenches, and large-diameter auger holes. Such samples are useful for determining inplace density and water content, and for other laboratory tests.

Figures 76, 77, and 78 show procedures commonly used in hand-cut block sampling. Cutting and trimming samples to desired size and shape requires extreme care, particularly when working with easily disturbed soft or brittle materials. The appropriate cutting tool should be used to prevent disturbance and cracking of the sample. Soft, plastic soils require thin, sharp knives, and sometimes a tight, thin, piano wire is advantageous. When climatic conditions are such that rapid drying occurs, wet cloths should be used or

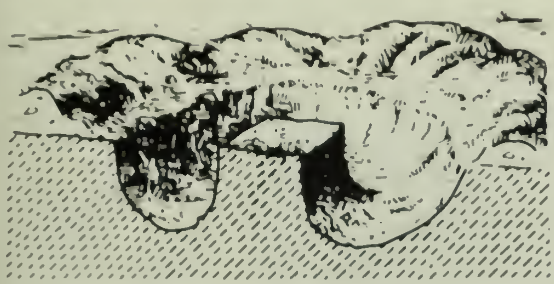
Figure 75. **Exploration for concrete aggregate including facilities for screening and weighing.**

other protective means provided while the sample is being cut. After the sample is cut and trimmed to the desired size and shape, it should be wrapped with a layer of cheesecloth and painted with warm sealing wax. Rubbing the surface with the bare hands will help to seal the pores in the wax. These operations constitute one layer. At least two additional layers of cloth and wax should be applied.

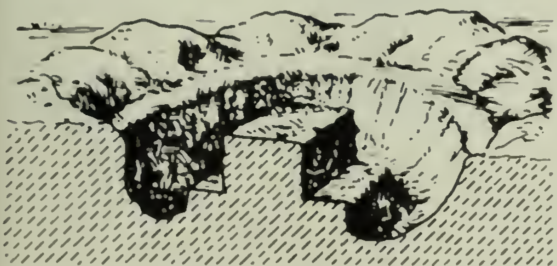
If a soil is easily disturbed, a firmly constructed wooden box with both ends removed should be placed over the sample before it is cut from the parent material and lifted for removal. Space between the sample and the walls should be packed with moist sawdust or similar packing material. The top cover of the box should then be placed over the packing material. After removal, the bottom side of the specimen should be covered with the same number of layers of cloth and wax as the other surfaces, and the bottom of the box placed over the packing material.

Samples may be of various sizes, the most common being 6- or 12-inch cubes. Cylindrical samples 6 to 8 inches in diameter and 6 to 12 inches long are frequently obtained as shown in figure 78. The metal cylinder shown in the figure may be used to confine the sampler for shipping. Otherwise, the same trimming and sealing procedures described above for boxed samples apply.

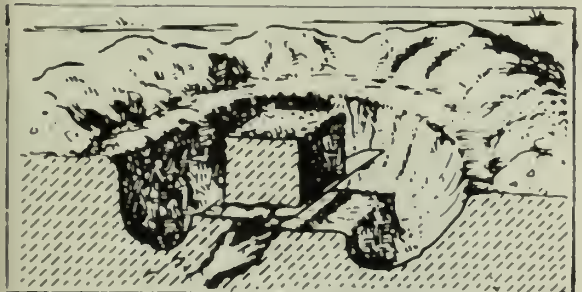
(c) *Rock Cores.*—Conventional rotary drilling and sampling were originally developed for drilling through hard and soft rocks. Samplers, called core barrels, will obtain cores of ⅞ inch to 2⅛ inches in diameter up to 20 feet long. There are two principal types of core barrels, single tube and double tube. They are shown in figure 79. The single tube is simpler in design and consists of a core-barrel head, core barrel, and attached coring bit, which has an annular groove that permits passage of drilling fluid pumped through the hollow drill rod. This design exposes the core to drilling fluid over its entire length, which results in serious core erosion of unconsolidated or weakly cemented materials. Therefore, the single tube is used primarily to sample hard, solid rock which requires a diamond drill bit.



1. Smooth ground surface and mark outline of sample.
2. Carefully excavate trench around sample.



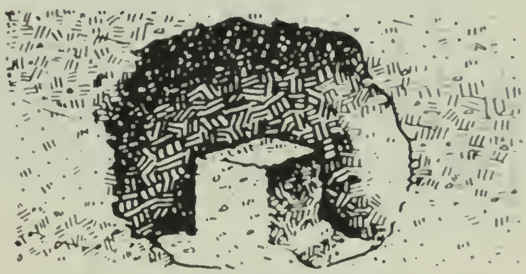
3. Deepen excavation and trim sides of sample to desired size with knife.



4. Cut sample from parent stratum, or encase sample in box before cutting if sample is easily disturbed.

(A)

1. Carefully smooth face surface and mark outline of sample.

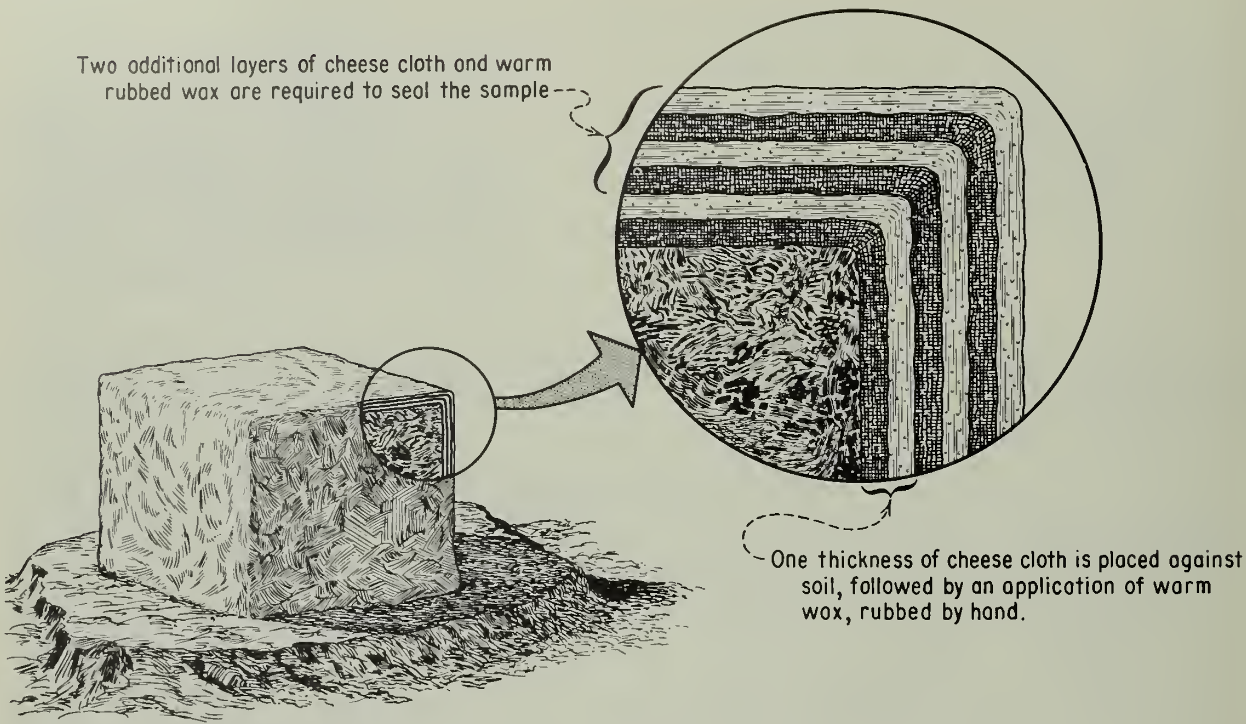


2. Carefully excavate around and in back of sample. Shape sample roughly with knife.

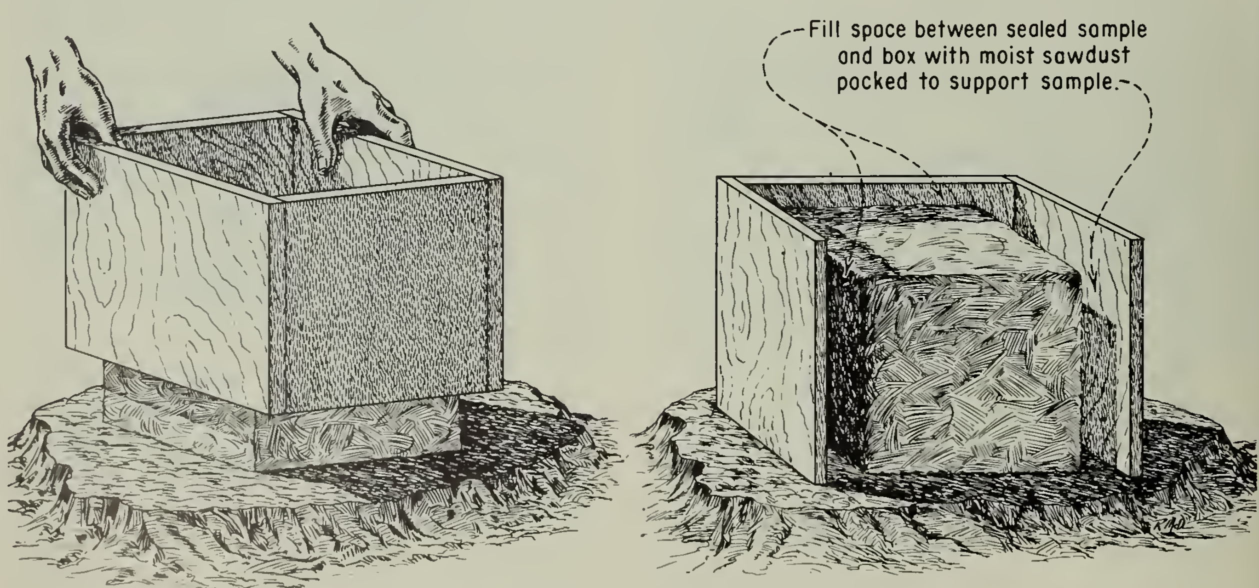
3. Cut sample and carefully remove from hole, or encase sample in box before cutting if sample is easily disturbed.

(B)

Figure 76. Initial steps to obtain a hand-cut undisturbed block sample from (A) bottom of test pit or level surface, (B) cut bank or side of test pit.

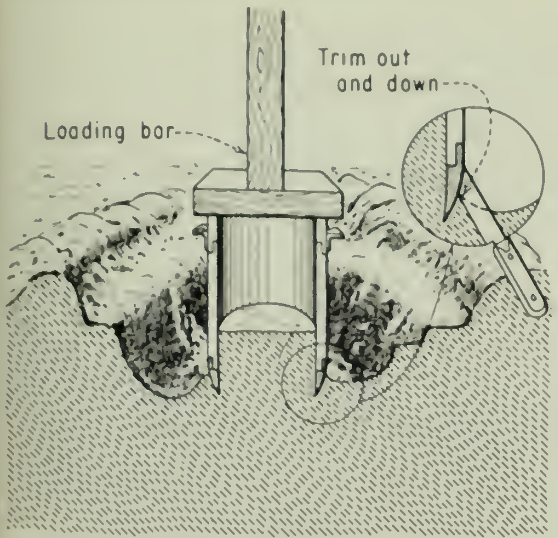


(A.) METHOD FOR SEALING HAND-CUT UNDISTURBED SAMPLES

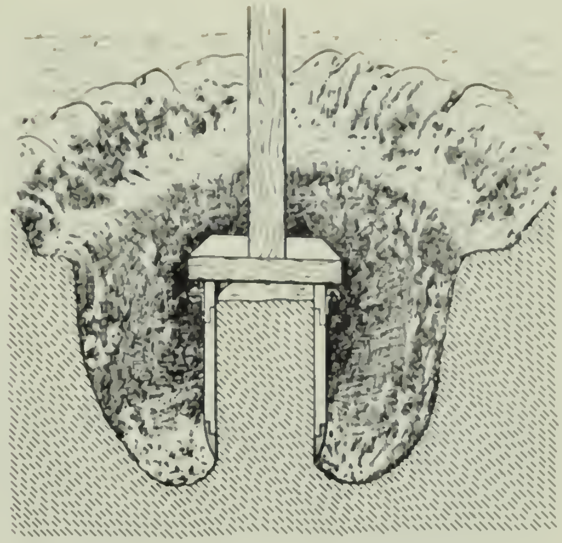


(B.) ENCASE EASILY DISTURBED SAMPLES IN BOX PRIOR TO CUTTING

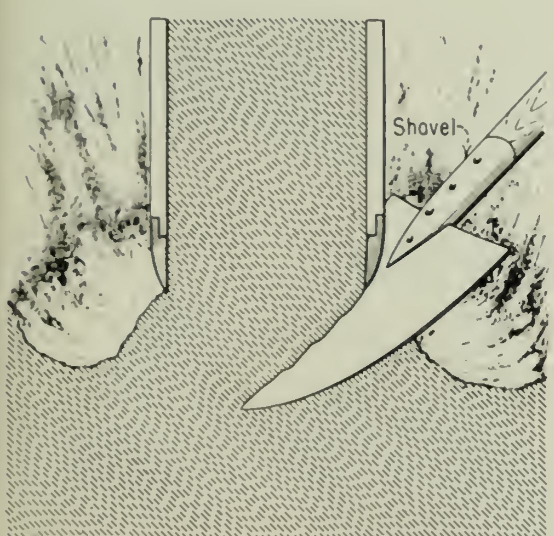
Figure 77. Final steps to obtain a hand-cut undisturbed block sample.



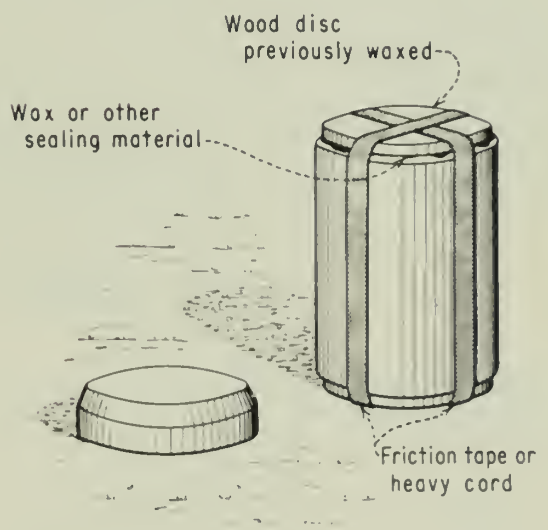
(A)
Level area and drive cylindrical sampler slightly into soil. Carefully excavate trench around cylinder and trim to cutting bit with knife.



(B)
Continue to drive sampler tube and excavate as shown.



(C)
Carefully cut sample from parent material as shown.



(D)
Seal sample to prevent moisture loss. Pack sample and container in excelsior or moist sawdust for shipment to the laboratory.

Figure 78. Method for obtaining a hand-cut undisturbed cylindrical sample.

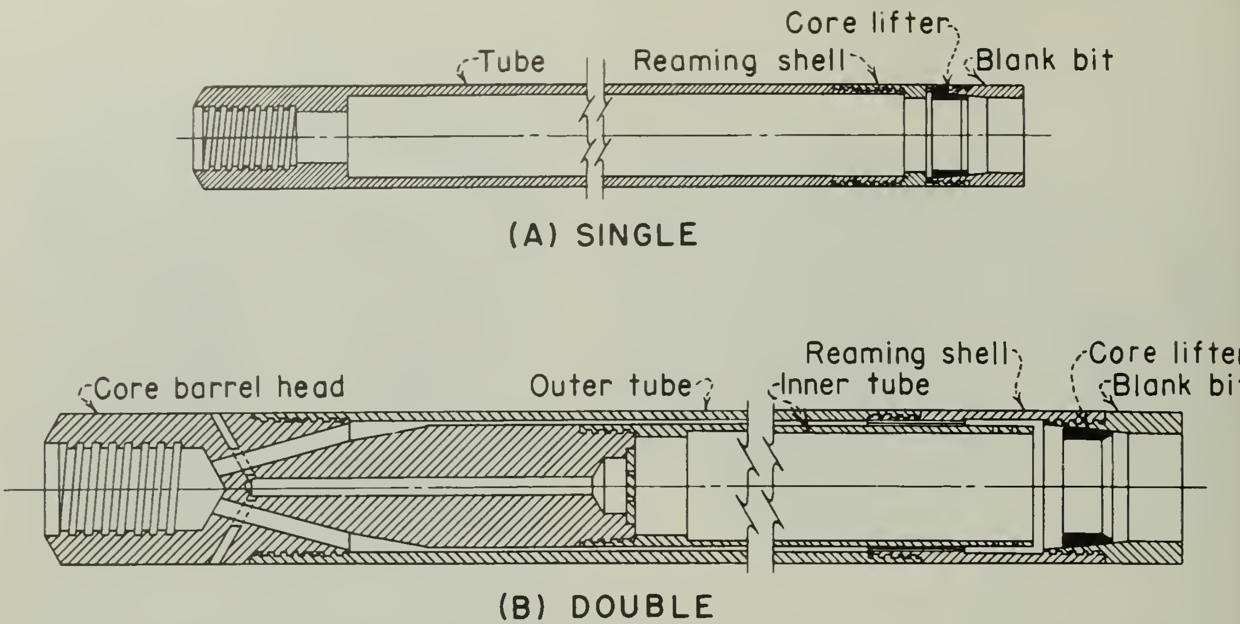


Figure 79. **Core barrels used for obtaining samples of rock.**

In addition to an outer rotating barrel, the double-tube core-barrel sampler provides an inner stationary barrel which protects the core from the drilling fluid and reduces the torsional forces transmitted to the core. It is used to sample soft or fractured rock and may be used to obtain cores in hard, brittle, or partially cemented soils, or cores of soft, weakly cemented rocks. For these materials, hardened metal drill bits and shorter core barrels (5 feet or 2½ feet in length) are used.

Figure 80 shows the dimensions of standard drilling casing, rod, and cores. Figure 81 shows a standard core box and illustrates the method of placing cores in the box to insure proper identification of each core sample.

H. LOGGING OF EXPLORATIONS

108. ***Identification of Holes.***—To insure completeness of the record and to eliminate confusion, test holes should be numbered in the order they are excavated, and the numbering series should be continuous through the various stages of investigation. If a hole is planned and programed, it is preferable to maintain the hole number in the record as "not drilled" or "abandoned" with an explanatory note rather than to reuse the hole number elsewhere. It is permissible, however, to move holes short distances and retain the program number where such moves are required by local conditions or by changes in engineering plans. When explorations cover several areas, such as alternative dam sites and different borrow areas, a new series of numbers for each site or borrow area should be used. A common practice is to start numbering each new area explored at an even 100.

Test holes are prefixed with a 1- or 2-letter designation to describe the type of exploration. The following letter designations are frequently used:

- DH Drill hole.
- AH Auger hole (hand).
- AP Auger hole (power).
- TP Open pit.
- T Trench.
- PR Penetration resistance hole.

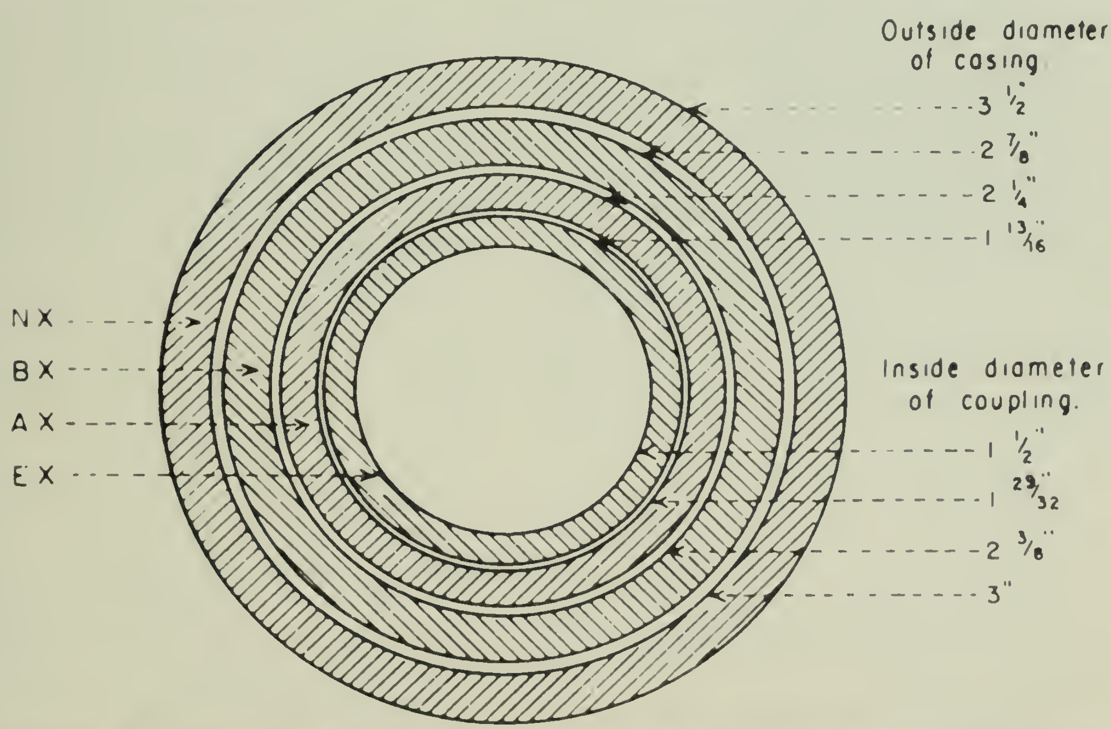
109. ***Log Forms.***—A log is a written record of the data concerning materials and conditions encountered in individual test holes. It provides the fundamental facts on which all subsequent conclusions are based, such as need for additional

NOMINAL DIMENSIONS

SIZE DESIGNATION		CASING O.D. Inches	CASING COUPLING		CASING BIT O.D. Inches	CORE-BARREL BIT O.D. Inches	DRILL ROD O.D. Inches	*APPROX. DIAMETER OF HOLE MADE BY CORE-BARREL BIT Inches	APPROX DIAMETER OF CORE Inches
CASING, CASING COUPLING, CASING BITS, C.B. BITS	ROD, ROD COUPLINGS		O.D. Inches	I.D. Inches					
EX	E	1 13/16	1 13/16	1 1/2	1 27/32	1 7/16	1 5/16	1 1/2	7/8
AX	A	2 1/4	2 1/4	1 29/32	2 3/16	1 27/32	1 5/8	1 7/8	1 1/8
BX	B	2 7/8	2 7/8	2 3/8	2 13/16	2 3/16	1 29/32	2 3/8	1 5/8
NX	N	3 1/2	3 1/2	3	3 9/16	2 15/16	2 3/8	3	2 1/8

* For a closer figure assume hole $\frac{1}{32}$ inch larger than bit

DIAMOND CORE DRILL CASING



SECTION THROUGH CASING COUPLINGS, AND NOMINAL DIMENSIONS

Figure 80. Nominal dimensions of diamond core drills.

Figure 81. **Arrangement of cores in a core box to insure identification of samples.**

exploration or testing, feasibility of the site, design treatment required, cost of construction, method of construction, and evaluation of structure performance. A log may represent pertinent and important information that is used over a period of years; it may be needed to delineate accurately a change of conditions with the passage of time; it may form an important part of contract documents; and it may be required as basic evidence in court in case of dispute. Each log, therefore, should be factual, accurate, clear, and complete. It should not be misleading. Log forms are used to provide the required information. Examples of logs for three types of exploratory holes are:

Geologic log of drill hole (fig. 82).—This form is suitable for all types of core borings which produce comparatively undisturbed samples.

Log of test pit or auger hole (fig. 83).—This form is suitable for all types of exploratory holes which produce complete but disturbed samples.

Penetration resistance log (fig. 84).—This form can be used for exploratory holes that test the soil in place.

Logs of trenches, tunnels, and shafts are best presented on drawings; these drawings should also contain the pertinent information outlined in figure 82.

The headings on the forms provide spaces for supplying identifying information as to project, feature, hole number, location, elevation, dates started and completed, and the name of the person responsible. Summary data such as depth to bedrock and to water table are useful. All of this information is important; any omissions should be justified. The body of the log form is divided into a series of columns covering the various kinds of information required according to the type of exploratory hole.

A log should always contain information on the size of the hole and on the type of equipment used for boring or excavating the hole. This should include the kind of drilling bit used on drill holes, a description of the penetrating equipment or type of auger used, or method of excavating test pits. The location from which samples are collected should be indicated on the logs, and the amount of material recovered as core should be expressed as a percentage of each length of penetration of the barrel. The logs should also show the extent and the method of support used as the

7-1337
(11-53)
BUREAU OF RECLAMATION

GEOLOGIC LOG OF DRILL HOLE

FEATURE EXAMPLE DAM PROJECT ECONOMY STATE WESTERN

HOLE NO 105 LOCATION Spillway Site COORDINATES N49,816; E50,151 GROUND ELEVATION 3486.3 ANGLE FROM VERTICAL 0°

BEGUN 8-2-56 FINISHED 8-2-56 DEPTH OF OVERBURDEN 3.6 ft TOTAL DEPTH 34.3 ft BEARING OF ANGLE HOLE -

DEPTH OR ELEV OF WATER TABLE See notes HOLE LOGGED BY John Day FOREMAN R. T. Roe

NOTES (On water table levels, water return, character of drilling etc)

Hole at 16.3', 4 pm, 8-2-56. 100% water return. 3½" Cs to 7.5'. Hole producing water.

Finished hole at 34.3'. 12mn, 8-2-56. 100% water return.

Pulled casing.

On 8-3-56, hole was producing water at a rate of 0.1 gpm.

TYPE AND SIZE OF HOLE: DP 3½" (0 to 7.5'); DMX (7.5' to 34.3')

CORE RECOVERY (%): 53, 92, 90, 71, 60, 66

PERCOLATION TESTS

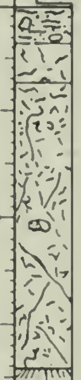
Depth (ft) From (P, Cs or Cm)	To	Loss in (G.P.M.)	Pressure (P.S.I.)	Length of test (min)
P9.3	Bottom 19.4	0.9	30	10
P19.0	29.0	1.2	30	10
P24.3	34.3	5.6	30	10

Note: Average core recovery from 7.5' to 34.3' was 19.2 feet out of 26.8 feet = 72%.

ELEVATION: 3486.3, 3482.7, 3478.8, 3452.0

DEPTH: 10, 20, 30, 40, 50, 60, 70, 80, 90

LOG (SAMPLES FOR TESTING)



CLASSIFICATION AND PHYSICAL CONDITION

0.0' - 3.6' Soil and rock reported. Drive sample contains brown soil and a few angular rock fragments with a few small roots.

3.6' - 7.5' Badly weathered andesite reported. Wash sample contains cuttings of weathered andesite.

7.5' - 34.3' ANDESITE, slightly weathered, light gray. Rock is in fair condition with a few harder zones throughout, but rock does not appear to improve materially with depth. Much of the rock can be scratched fairly easy with a knife. Some of the rock is soft enough so that it can be crumbled in the hands. The most notable of these zones is between 8' and 11'. Most of the rock cored fairly good, but much of the core is quite broken. Broken zones where most of the core is smaller than 2" are from 7.5' to 11.3', 14.7' to 15.3', 17.0' to 19.9', 20.7' to 20.9', 22.3' to 24.6', and 28.5' to 34.3'. Other than for the broken zones, the lengths of core ranged from about 2" to about 15". Rock contains numerous small feldspar phenocrysts that give material a medium grained appearance. Much of the feldspar is slightly altered and cloudy. There are numerous joints throughout the core and all joint surfaces are thinly coated with dark brown to tan material. Several small inclusions of soft reddish material are scattered throughout the core with one length from 19.6' to 19.9' that can be easily scraped with a knife.

34.3' Bottom of hole.

EXPLANATION

DP = Drive Pipe

Type of hole	D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed	P = Packer, Cm = Cemented, Cs = Bottom of casing
Approximate size of hole (X-series)	Ex = 1½", Ax = 1⅞", Bx = 2⅜", Nx = 3"
Approximate size of core (X-series)	Ex = ⅞", Ax = 1⅛", Bx = 1⅝", Nx = 2⅛"
Outside diameter of casing (X-series)	Ex = 1 13/16", Ax = 2¼", Bx = 2⅞", Nx = 3½"
Inside diameter of casing (X-series)	Ex = 1½", Ax = 1 29/32", Bx = 2⅜", Nx = 3"

CORE LOSS / CORE RECOVERY

ANGLE HOLE ☐ VERTICAL HOLE ☒

HOLE NO 105

Figure 82. Geologic log of drill hole in a spillway foundation.

LOG OF TEST PIT OR AUGER HOLE
FOR BORROW AND FOUNDATION INVESTIGATIONS

Feature EXAMPLE DAM Project ECONOMY Hole No TP 455
Area Designation BORROW AREA 4 Coordinates N. 16,500 E. 6,910 Ground Elevation 669.8 Depth To Ground Water Level * 21.9 FT.
Method of Excavation HAND DUG PIT Approximate Dimensions of Hole 4 X 5 FEET Dates of Excavation APRIL 5-12, 1949 Hole Logged By JOHN DOE

CLASSIFICATION SYMBOL: LETTER	CLASSIFICATION SYMBOL: GRAPHIC	DEPTH (FEET)	SIZE AND TYPE OF SAMPLE TAKEN	CLASSIFICATION AND DESCRIPTION OF MATERIAL (SEE CHART - "UNIFIED SOIL CLASSIFICATION"; GIVE GEOLOGIC AND IN-PLACE DESCRIPTION FOR FOUNDATION INVESTIGATIONS)	PERCENTAGE OF COBBLES AND BOULDERS **: VOLUME OF HOLE SAMPLED (CUBIC FEET)	WEIGHT OF 3 TO 5-INCH SAMPLED (LBS.)	PERCENTAGE BY VOLUME OF 3 TO 5-INCH***	WEIGHT OF PLUS 5-INCH SAMPLED (LBS.)	PERCENTAGE BY VOLUME OF PLUS 5-INCH***
ML		2	75 lbs. sack	Silt; slightly organic with some alfalfa and weed roots, dark brown; small amount of fine sand; dry; non-plastic.	40	0	0	0	0
CL		8.5	175 lbs sack	Lean clay; moderately plastic; high dry strength; about 25 percent sand and gravel up to 3/4 inch; most of gravel is shale; brown; dry.	130	0	0	0	0
ML-MH		16	200 lbs. sack	Micaceous silt; moderate amount of very fine sand, no gravel; noticeable mica flakes; tan; very slight plasticity; dry.	150	0	0	0	0
ML		18	90 lbs. sack	Silt; similar to material 8.5 feet to 16 feet but contains about 20 percent shaly gravel up to 1 inch; dry.	40	0	0	0	0
GW	GWL	22	155 lbs. (-3 inch) sack	Gravel-sand mixture; well graded; very small amount of silty fines; gravel mostly hard, subrounded, 8 inch maximum size; about 50 percent gravel. Dry above water table; river terrace gravel.	80	2430	19.1	1080	8.5

REMARKS: Density - in-place test at 8 feet: dry density = 89.4 lbs. per cu. ft., water content = 8.9%. Bulk specific gravity of cobbles and boulders is 2.55, by displacement method. Ground water level reading made April 14, 1949.

NOTES: Record water test and density test data, if applicable, under remarks.
* Record after water has reached its natural level; give date of reading adjacent to graphic symbol or in remarks.
** Applicable only to borrow pits and to foundations which are potential sources of construction materials

$$*** \quad \frac{(\text{Lbs of rock sampled})\ 100}{(\text{Bulk specific gravity of rock})\ 62.4\ (\text{Cubic feet of hole sampled})}$$

Record bulk specific gravity in Remarks, stating how obtained (measured or estimated)

Figure 83. Example of log of test pit in a borrow area.

SUBSURFACE EXPLORATION – PENETRATION RESISTANCE AND LOG

FEATURE	Small Dam	GROUND ELEVATION	5022 2	HOLE NUMBER	PR 4
PROJECT	Example	ELEV WATERTABLE	5010 0	LOCATION	Foundation
STATE	New Mexico	DATE WATERTABLE GAGED	4-29-55	COORDINATES	N 62,500 E 122,520
FOREMAN	J Simon	WEIGHT OF HAMMER	140 lbs	TOTAL DEPTH OF HOLE	15 9 FT
HOLE LOGGED BY	K R Clark	HEIGHT OF DROP	30 in	DATE BEGUN	4-28-55 FINISHED 4-28-55

NOTES Type and size of hole Type of bit or spoon Loss of drilling water	NO OF BLOWS	DESCRIPTION OF MATERIAL
Nx Casing Standard 2" OD split barrel sampler 30" long Hole cleaned by fishtail bit or rotary drill	3	05-15' SANDY SILT Trace of clay, dry, calcareous Slightly plastic Predominantly minus No. 200 (ML)
	7	20-31' SILTY CLAY Clay, sand, gravel and silt, moist, calcareous Medium plasticity Predominantly minus No 200 (CL)
	8	36-46' SILT CLAY Clay, silt, some sand, moist, calcareous Medium plasticity Predominantly minus No 200 (CL)
	17	51-62' SILTY CLAY WITH GRAVEL Clay, silt, sand and gravel, moist, calcareous Medium plasticity Predominantly minus No 200 (CL)
	9	67-77' SILTY CLAY Clay, silt, and sand, moist, calcareous Medium plasticity Predominantly minus No 200 (CL)
	7	82-97' CLAYEY SILT Some sand, moist, calcareous Low plasticity Predominantly minus No 200 (ML-CL)
	4	97-107' Recovery is probably caved material Water table 11 to 12'
	7	112-122' SILTY FINE SAND Some clay, organic, wet, calcareous. Slightly plastic Predominantly No 100 and finer (SM)
	14	127-137' SILTY FINE SAND Some clay, slightly organic, wet, calcareous. Slightly plastic. Predominantly No 100 and finer. (SM)
	5	142-153' No recovery
	4	158-169' FINE SAND Some silt and small amount of gravel, wet, calcareous Nonplastic Predominantly No 50 to 200 (SP)

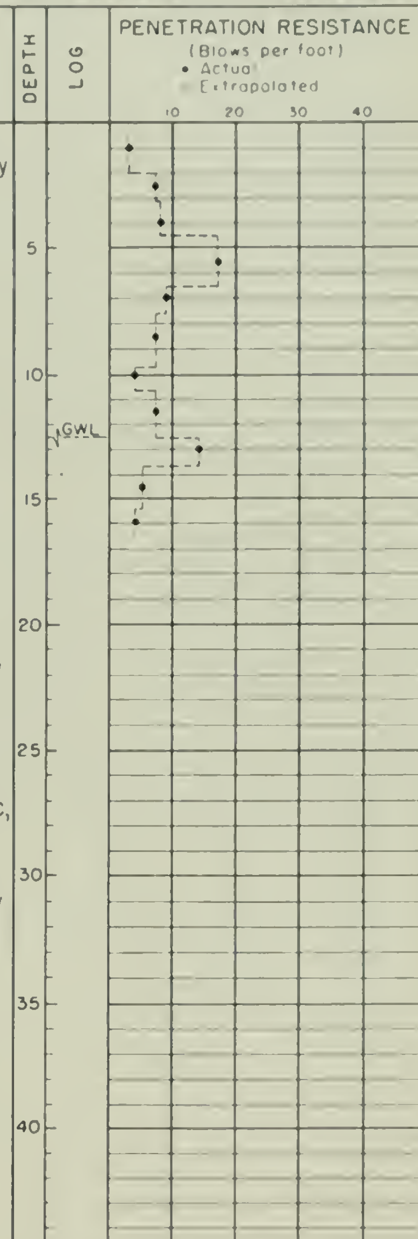


Figure 84. Example of penetration resistance and log obtained in standard penetration test.

hole is deepened, such as size and depth of casing, location and extent of grouting if used, type of drilling mud, or type of cribbing in test pits.

Information on the presence or absence of water levels and comments on the reliability of these data should be given on all logs. The date measurements are made should be recorded, since water levels fluctuate seasonally. Water levels should be recorded periodically from the time water is first encountered and as the test hole is deepened. Perched water tables and water under artesian pressure are important to note. The extent of water-bearing members should be noted and areas where water is lost as the boring proceeds should be reported, since subsequent work on the hole may preclude duplicating such information. The log should contain information on the water tests made at intervals, as described in section 113. Since it may be desirable to maintain periodic records of water level fluctuations in drilled holes, it should be determined whether this is required before abandoning and plugging the exploratory hole.

Where cobbles and boulders are encountered in explorations for sources of embankment materials, it is important to determine their percentage by volume. The log form for test pit or auger hole (fig. 83) includes a method for obtaining the percentage by volume of 3- to 5-inch rock and rock over 5 inches in size. The method involves weighing the rock, converting weight to solid volume of rock, and measuring the volume of hole containing the rock. This determination can be made either on the total volume of stratum excavated or on a representative portion of the stratum by means of a sampling trench which is described in section 106.

For test holes that penetrate less than 25 feet of potential borrow material, a statement should be made under "Remarks" in the log giving the reason for stopping the hole. For all other types of holes, a statement should be made at the end of the log that the work was completed as required or a statement explaining why the hole was abandoned. Material should not be described as bedrock, ledge rock, slide material, or similar interpretative terminology unless the exploration actually penetrated such conditions and samples were collected to substantiate these conclusions. Terminating statements similar to the following would be considered satisfactory: Hole eliminated due to lack of funds; hole caved in; depth limited by capacity of equipment; encountered water; unable to penetrate hard material in bottom of hole.

110. *Description of Soils.*—The person logging exploratory holes should be able to identify soils according to the Unified Soil Classification System. The description of a soil in a log should include its typical name, followed by pertinent descriptive data, as listed in table 11. After the soil is described, it should be placed in the appropriate soil classification group by use of letter symbols. These group symbols represent a variety of soils having certain common characteristics; hence, by themselves they are not sufficient to describe a particular soil. Boundary classifications (two sets of symbols separated by a hyphen) should be used when the soil does not fall clearly into one of the groups but has strong characteristics of both groups.

TABLE 11.—*Description of soils*

Items of descriptive data	Borrow		Foundation	
	Coarse-grained soils	Fine-grained soils	Coarse-grained soils	Fine-grained soils
Typical name (examples are shown in classification chart)	XX	XX	XX	XX
Approximate percentages of gravel and sand	X		X	
Maximum size of particles (including cobbles and boulders)	XX		X	
Shape of the coarse grains—angularity	X		X	
Surface conditions of the coarse grains—coatings	X			
Hardness of the coarse grains—possible breakdown into smaller sizes	X		X	
Color (in moist condition for fine-grained soils)	X	X	X	X
Moisture and drainage conditions (dry, moist, wet, saturated)	XX	XX	XX	XX
Organic content	X	X	X	X
Plasticity (of fine fraction in coarse-grained soils; degree and character for fine-grained soils)	X	XX	X	XX
Amount and maximum size of coarse grains		X		X
Structure (stratification, etc., give dip and strike; honey comb flocculent, root holes)			XX	XX
Cementation—type			XX	XX
Degree of compactness—loose or dense (excepting clays)			XX	XX
Consistency in undisturbed and remolded states (clays only)				XX
Local or geologic name	X	X	X	X
Group symbol	XX	XX	XX	XX

The purposes for which soils are investigated for small dams can be divided into two categories: (1) Borrow materials for embankments or for backfill, and (2) foundations for the dam and appurtenant structures. The emphasis of various features to be described depends on which of the categories is involved. For many structures large quantities of soil must be excavated to reach a desired foundation. In the interests of economy, maximum use should always be made of this excavated material in the construction of embankments and for backfill. A foundation area, therefore, often becomes a source of materials, and investigation of the area must take into account this dual purpose. Descriptions of soils encountered in such explorations should contain the essential information required both for borrow materials and for foundation soils.

Soils that are potential sources of borrow material for embankments must be described adequately in the log of the exploratory test pit or auger hole. Since these materials are destined to be disturbed by excavation, transportation, and compaction in the fill, their structure is less important than the amount and characteristics of the soil constituents. However, the recording of their natural water condition is important. Very dry borrow materials require the addition of large amounts of moisture for compaction control, and very wet soils containing appreciable fines may require extensive processing in order to be usable. For simplicity the natural water content of borrow soils should be reported as either *dry*, *moist*, or *wet*. A soil that is reported dry should be one that will definitely require the addition of moisture in order to compact it properly in a fill. A soil should be reported as being moist if it is reasonably close to the Proctor optimum water content. Soils that are reported as wet should be obviously well beyond the optimum water content. Borrow-pit holes are logged so as to indicate divisions between soils of different classification groups. However, within the same soil group significant changes in moisture should be logged.

When soils are being explored as foundations for dams and appurtenant works, their natural structure, compactness, and moisture content are of outstanding importance. Logs of foundation explorations, therefore, must emphasize the inplace condition of a soil in addition to describing its constituents. The natural state of foundation soils is significant because bearing capacity and settlement under load vary tremendously with the consistency or compactness of the soil. Therefore, information that a clay soil is hard and dry, or soft and moist is important. Changes in consistency of foundation soils due to moisture changes under operating conditions must be considered in the design. Correct classification is needed so that the effect of these moisture changes on foundation properties can be predicted.

Table 11 lists data that are needed to describe soils for borrow material and for foundations. Under each of these categories, the information desired for coarse-grained soils and for fine-grained soils is indicated by an X. All of these descriptive data are not always needed. Judgment should be used to include pertinent information, to avoid negative information, and to eliminate repetition. The items indicated by XX should always be reported. Examples of soil descriptions are given in the soil classification chart, figure 38, and in the examples on the log forms, figures 82, 83, and 84.

111. *Description of Rock Cores.* The basic objective of describing rock cores is to provide a concise record of the important geological and physical characteristics of the core materials. The description should be prepared preferably by a geologist; its usefulness will be controlled largely by the individual's experience in logging rock core for engineering purposes and his knowledge of geology. Thus, an experienced individual will include some seemingly minor core features or conditions which he knows have engineering significance and exclude other geologic features having only academic interest.

Description of the rock core should include its typical rock name followed by data on its lithologic and structural features, physical condition including alteration, and any special geologic, mineralogic, or physical features pertinent to interpretation of the subsurface conditions. Classification of rocks is given in part D of this chapter. Attention should be given to (1) the attitude and severity of joints, seams or fractures and whether open or filled, as well as to evidence of shearing, crushing, or faulting; (2) planes of bedding, lamination or layering and the ease of splitting along such planes; (3) color, grain size and shape, and (in sedimentary rocks like sandstone) the mineralogy of the grains and cementing material as well as the extent to which the cementing material

occupies the intergrain spaces; and (4) the degree of alteration or weathering and hardness of the rock. In the latter case supplementary phrases such as "breaks with sharp hammer blow," "crumbles easily in the fingers," or "hard as common brick" are helpful. Estimates of the average length of core pieces in successive sections of the hole aid in calling attention to changes in formations or rock conditions in the hole not otherwise recognizable but nonetheless useful in evaluating subsurface conditions in the engineering sense.

The purpose of the drilling and logging is to secure evidence of the "inplace" condition of the rock; therefore, care should be taken to note any core condition or damage due to the type of drill bit or core barrel used or to improper conduct of the drilling process. Such factors may have a marked effect on the amount and condition of the core recovered, particularly in soft, friable, or severely fractured rock.

Adequate logs or descriptions of rock core can be prepared by a reasonably experienced individual solely through visual or "hand specimen" examination of the core with the occasional aid of simple field tests. Detailed microscopic or laboratory testing to define rock type or mineralogy is generally necessary only in special cases. Figure 81 shows how rock cores obtained from a borehole are arranged for logging.

I. FIELD AND LABORATORY TESTS

112. ***General.***—Of the great variety of field and laboratory tests that have been used in the design of dams, only those applicable to the simplified design procedures used in this text are described here. In addition to the quantitative data (the number of blows per foot) obtained during the standard penetration borings described in section 103, two other field tests which obtain values for the natural ground and which are applicable in exploring foundations for small dams are: (1) Permeability tests and (2) density-in-place tests. The latter test is used also in borrow areas to determine shrinkage factors between excavation and embankment yardages.

Descriptions of laboratory tests on soils are limited to those required to verify soil classifications and to determine compaction characteristics for comparison with design assumptions made from data of table 6 (sec. 89) and for correlation with construction control tests given in appendix E. The descriptions of the tests are intended to furnish a general knowledge of their scope. For detailed test procedures reference should be made to the Bureau of Reclamation Earth Manual [7] or to the ASTM Designation [9] noted for several of the tests.

The laboratory quality tests of riprap and concrete aggregate, commonly used in specifications for these materials, are described to afford an understanding of the significance of those tests. Details of test procedures can be found in the Bureau of Reclamation Concrete Manual [8] and in the referenced ASTM Designation [9].

113. ***Field Permeability Tests.***—(a) *General.*—Approximate values of permeability of individual strata penetrated by borings can be obtained by making water tests in the holes. The reliability of the values obtained depends on the homogeneity of the stratum tested and on certain restrictions of the mathematical formulas used. However, if reasonable care is exercised in adhering to the recommended procedures, useful results can be obtained during ordinary boring operations. Use of the more precise methods of determining permeability by pumping from wells with a series of observation holes to measure drawdown of the water table or by pumping-in tests using large-diameter perforated casing is considered unnecessary for the design of small dams.

The tests described below are of the pumping-in type; that is, they are based on measuring the amount of water accepted by the ground through the open bottom of a pipe or through an uncased section of the hole. These tests are invalid and may be grossly misleading unless clear water is used. The presence of even small amounts of silt or clay in the added water will plug up the test section and give permeability results that are too low. By means of a settling tank or a filter, efforts should be made to assure that only clear water is used. It is desirable for the temperature of the added water to be higher than ground-water

temperature so as to preclude the creation of air bubbles in the ground which may greatly reduce the acceptance of water.

(b) *Open-End Tests.*—Figure 85 (A) and (B) shows a test made through the open end of a pipe casing which has been sunk to the desired depth and which has been carefully cleaned out just to the bottom of the casing. When the hole extends below the ground-water table, it is recommended that the hole be kept filled with water during cleaning and especially during withdrawal of tools to avoid squeezing of soil into the bottom of the pipe. After the hole is cleaned to the proper depth, the test is begun by adding clear water through a metering system to maintain gravity flow at a constant head. In tests above the water table (fig. 85(B)) a stable, constant level is rarely obtained and a surging of the level within a few tenths of a foot at a constant rate of flow for about 5 minutes is considered satisfactory.

If it is desired to apply pressure to the water entering the hole, the pressure, in units of head, is added to the gravity head as shown in figure 85 (C) and (D). Measurements of constant head, constant rate of flow into the hole, size of casing pipe, and elevations of top and bottom of casing are recorded. The permeability is obtained from the following relation determined by electric analogy experiments:

$$K=\frac{Q}{5.5rH} \tag{2}$$

where:

K = permeability,
Q = constant rate of flow into the hole,
r = internal radius of casing, and
H = differential head of water.

Any consistent set of units may be used. For convenience equation (2) can be written:

$$K \text{ (in feet per year)} = C_1 \frac{Q \text{ (in gallons per minute)}}{H \text{ (in feet)}}$$

Values of C_1 vary with size of casing as follows (see fig. 82):

Size of casing	EX	AX	BX	NX
C_1	204,000	160,000	129,000	102,000

The value of H for gravity tests made below water table is the difference in feet between the level of water in the casing and the ground-water level. For tests above water table, H is the depth of water in the hole. For pressure tests the applied pressure in feet of water (1 p.s.i. = 2.31 feet) is added to the gravity head to obtain H.

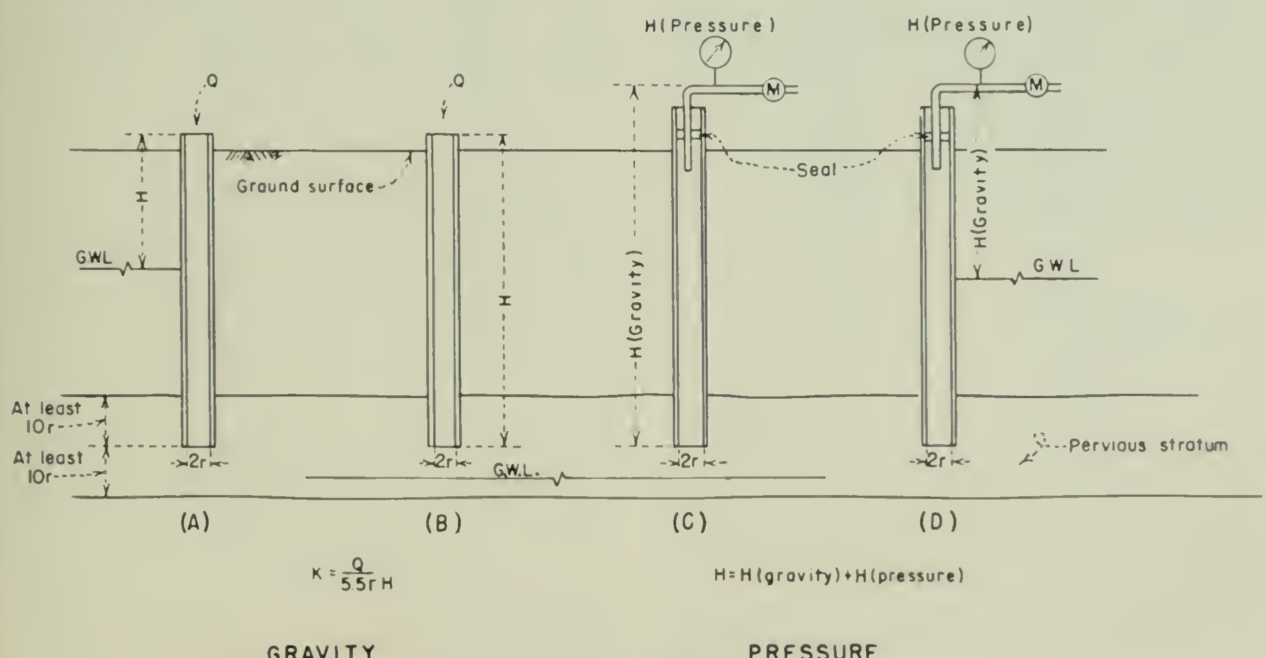


Figure 85. An open-end pipe test for soil permeability which can be made in the field.

For the example shown in figure 85(A):

Given:

NX casing,

Q=10.1 gallons per minute,

H=21.4 feet,

$$K=C_1\frac{Q}{H}=\frac{(102,000)(10.1)}{21.4}$$

=48,100 feet per year.

For the example shown in figure 85(D):

Given:

NX casing,

Q=7 gallons per minute,

H (gravity)=24.6 feet,

H (pressure)=5 p.s.i.=5×2.31

=11.6 feet of water.

Then H=24.6+11.6=36.2 feet, and

$$K=C_1\frac{Q}{H}=\frac{(102,000)(7)}{36.2}$$

=19,700 feet per year.

(c) *Packer Tests.*—Figure 86 shows a permeability test made in a portion of a drill hole below the casing. This test can be made both above and below the water table provided the hole will remain open. It is commonly used for pressure testing of bedrock using packers, but it can be used in unconsolidated materials where a top packer is placed just inside the casing.

The formulas for this test are:

$$K=\frac{Q}{2\pi LH}\log_e\frac{L}{r}, L\geqq 10r \qquad (3)$$

$$K=\frac{Q}{2\pi LH}\sinh^{-1}\frac{L}{2r}, 10r>L\geqq r \qquad (4)$$

where:

K=permeability,

Q=constant rate of flow into the hole,

L=length of the portion of the hole tested,

H=differential head of water,

r=radius of hole tested,

$\log_e$=natural logarithm, and

$\sinh^{-1}$=arc hyperbolic sine.

These formulas have best validity when the thickness of the stratum tested is at least $5L$, and are

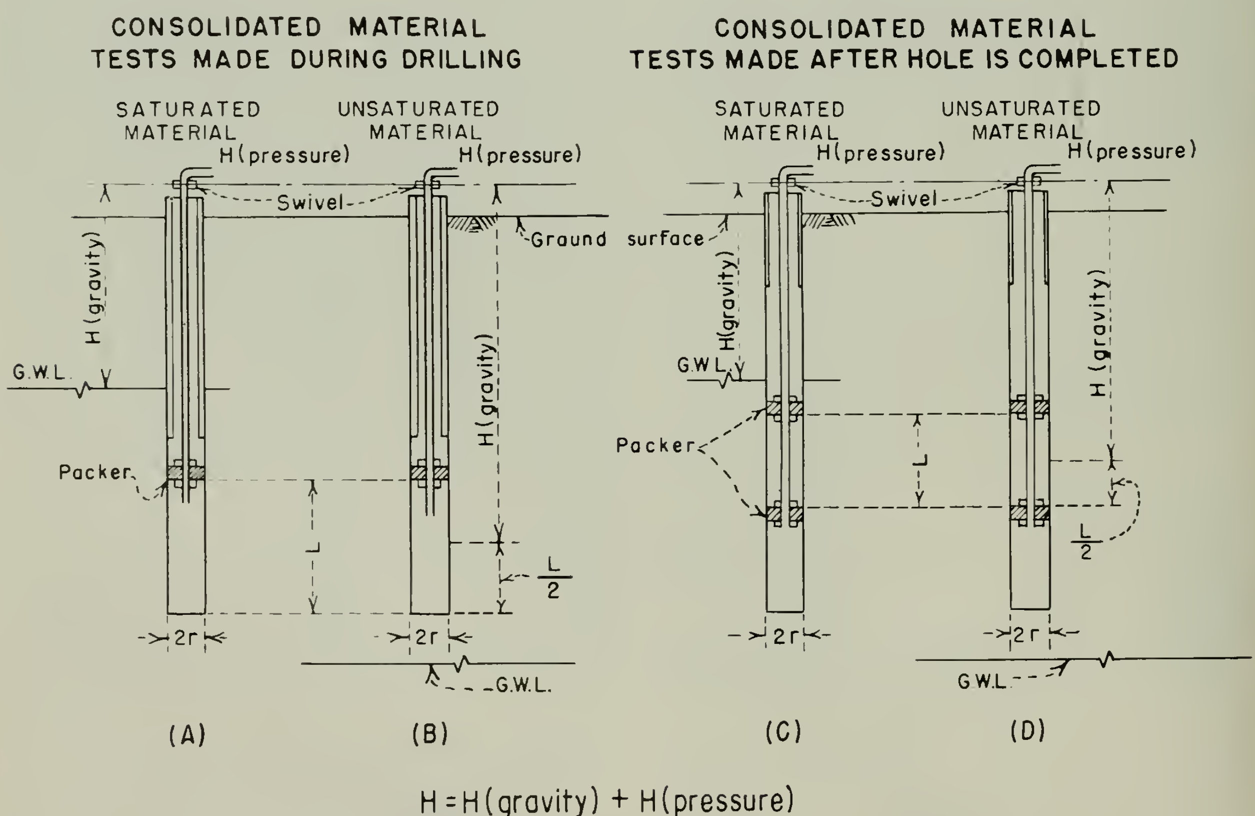


Figure 86. **The packer test for soil permeability.**

considered to be more accurate for tests below ground-water table than above it.

For convenience, the formulas can be written:

$$K \text{ (feet per year)} = C_p \frac{Q \text{ (gallons per minute)}}{H \text{ (feet)}}$$

where H is head of water in feet acting on the test length. Where the test length is below the water table, H is the distance in feet from the water table to the swivel plus applied pressure in units of feet of water. Where the test length is above the water table, H is the distance in feet from the center of the length tested to the swivel plus the applied pressure in units of feet of water. For gravity tests (no applied pressure) measurements for H are made to the water level inside the casing (usually the level of the ground).

Values of C_p are given in the following table for various lengths of test section and hole diameters:

Length of test section in feet, L	Diameter of test hole (see fig. 82)			
	EX	AX	BX	NX
1	31,000	28,500	25,800	23,300
2	19,400	18,100	16,800	15,500
3	14,400	13,600	12,700	11,800
4	11,600	11,000	10,300	9,700
5	9,800	9,300	8,800	8,200
6	8,500	8,100	7,600	7,200
7	7,500	7,200	6,800	6,400
8	6,800	6,500	6,100	5,800
9	6,200	5,900	5,600	5,300
10	5,700	5,400	5,200	4,900
15	4,100	3,900	3,700	3,600
20	3,200	3,100	3,000	2,800

The usual procedure is to drill the hole, remove the core barrel or other tool, seat the packer, make the test, remove the packer, drill the hole deeper, set the packer again to test the newly drilled section, and repeat the test (see fig. 86(A)). If the hole stands without casing, a common procedure is to drill it to final depth, fill with water, surge it, and bail it out. Then set two packers on pipe or drill stem as shown in figure 86 (C) and (D). The length of packer when expanded should be five times the diameter of the hole. The bottom of the pipe holding the packer must be plugged and its perforated portion must be between the packers. In testing between two packers, it is desirable to start from the bottom of the hole and work upward.

Example for figure 86(A):

Given:

NX casing set to depth of 5 feet,

$Q = 2.2$ gallons per minute,

$L = 1$ foot,

H (gravity) = distance from ground-water level to swivel = 3.5 feet,

H (pressure) = 5 p.s.i. × 2.31 = 11.55 feet of water,

$H = H\text{(gravity)} + H\text{(pressure)} = 15.1$ feet.

From table, $C_p = 23{,}300$

$$K = C_p \frac{Q}{H} = \frac{(23{,}300)(2.2)}{15.1} = 3{,}400 \text{ feet per year.}$$

114. *Density-in-Place Tests.*—(a) *Sand Density Method.*—This method is used to determine the inplace density in a foundation, a borrow area, or a compacted embankment by excavating a hole from a horizontal surface, weighing the material excavated, and determining the volume of the hole by filling it with calibrated sand. A water content determination on a sample of the excavated soil enables the dry density in the ground to be calculated. Various devices using balloons and water or oil have been used to measure the volume of the hole, but the sand method is most common.

About 100 pounds of clean, air-dry, uniform sand passing the No. 16 sieve and retained on the No. 30 sieve has been found to be satisfactory. Clean "blow sand" or dune sand is suitable. When large test holes are used in gravelly soils, coarse sand having rounded particles and passing the No. 4 sieve and retained on the No. 8 sieve is recommended. The sand is calibrated by pouring it into a container of known volume of approximately the size and shape of the type of excavation to be used, weighing it, and calculating its placed unit weight.

At the location to be tested, all loose soil is removed from an area 18 to 24 inches square and the surface is leveled. A working platform supported at least 3 feet from the edge of the test hole should be provided when excavating in soils that may deform and change the dimensions of the hole due to the weight of the operator. An 8-inch-diameter hole 12 to 14 inches deep is satisfactory for cohesive soils which contain little or no gravel. A hole about 12 inches in diameter at the surface, tapering down to about 6 inches at a depth of 12 to 14 inches, is needed for gravelly soils.

A steel or wooden template with the proper sized hole is placed on the ground and the excavation is carefully made with an auger or other handtools. All material taken from the hole is placed in an airtight container for subsequent weighing. To avoid loss of moisture the cover should be kept on the container except when in use; and in hot, dry climates shade for the test area and a moist cloth over the container should be provided.

The volume of the hole is determined by carefully filling it with calibrated sand using the sand-pouring device shown in figure 87 or similar method. The weight of sand used to fill the hole is determined by subtracting the final weight of sand and container (plus the calculated weight of sand occupying the small space in the template), from the initial weight. The volume of the sand (and of the hole) is calculated from the known unit weight of calibrated sand.

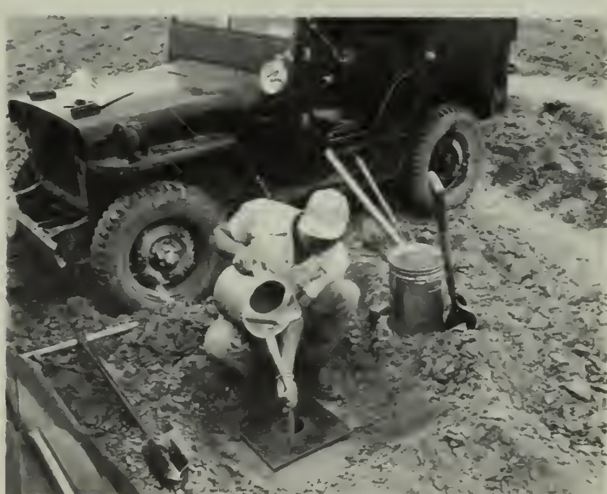


Figure 87. **Determining density of rolled embankment material by replacement with a sand of known density. 794–701–401.**

The inplace wet density of the soil is the weight of the soil removed from the hole divided by the volume of the hole. For soils containing no gravel, a representative moisture sample is taken and the water content is determined (see section 115 for water content test). The inplace dry density is then calculated.

For soils containing gravel sizes, the wet density of the total material is determined as in the foregoing. In the laboratory the gravel particles are separated from the soil, and their weight and solid volume are determined and subtracted from the total weight of material and volume of the hole, respectively, to obtain the wet density of the minus No. 4 fraction of the soil. This is converted to dry density by a water content determination. The field and laboratory procedures used for density inplace tests are shown in figure 88.

(b) *Method for Dry, Gravel-Free Soils.*—It is often necessary to determine the inplace dry density and water content of fairly deep foundations of cohesive soils above the water table. The sand density method given in (a) above requires a test pit or large-diameter auger hole to gain access to the foundation. The following simple method has been used successfully to obtain inplace density in stages of depth in foundations and borrow areas of dry gravel-free soils with the use of a hand auger.

A platform should be built on which the investigator can stand without bearing on the soil within 2 feet of the hole he is to auger. A system of sills and joists covered by decking containing a hole about 12 inches in diameter in the center will suffice. Such a device will preclude squeezing of the upper portion of the hole. The procedure is to start a hole with an 8-inch-diameter post-hole auger, penetrating the soil for a distance of between 6 inches and 1 foot, depending on the probable depth of stripping. The soil removed is discarded, and the depth from the surface of the ground to the apex of the cone at the bottom of the hole is measured to within 0.01 foot. The hole is then deepened with the auger to a depth of 3 feet or to any apparent change in soil structure, whichever occurs first; and the soil removed is placed on a clean canvas, sampled for water content, and weighed. The depth from the bottom of the hole to the top of the ground is carefully measured again to the nearest 0.01 foot, and the diameter of the hole is measured at about 1 foot below the surface of the ground.

The volume of the hole sampled is the difference between the two depths measured, multiplied by the area of the hole as computed from the measured diameter. Thus, the wet density and the dry density can be determined for each tested depth below stripping. The tests can be continued to the limit of the practicable hand-auger depth, which is about 20 feet for an 8-inch-diameter auger and which can be extended by use of a tripod to aid in removing the auger from the hole. Since

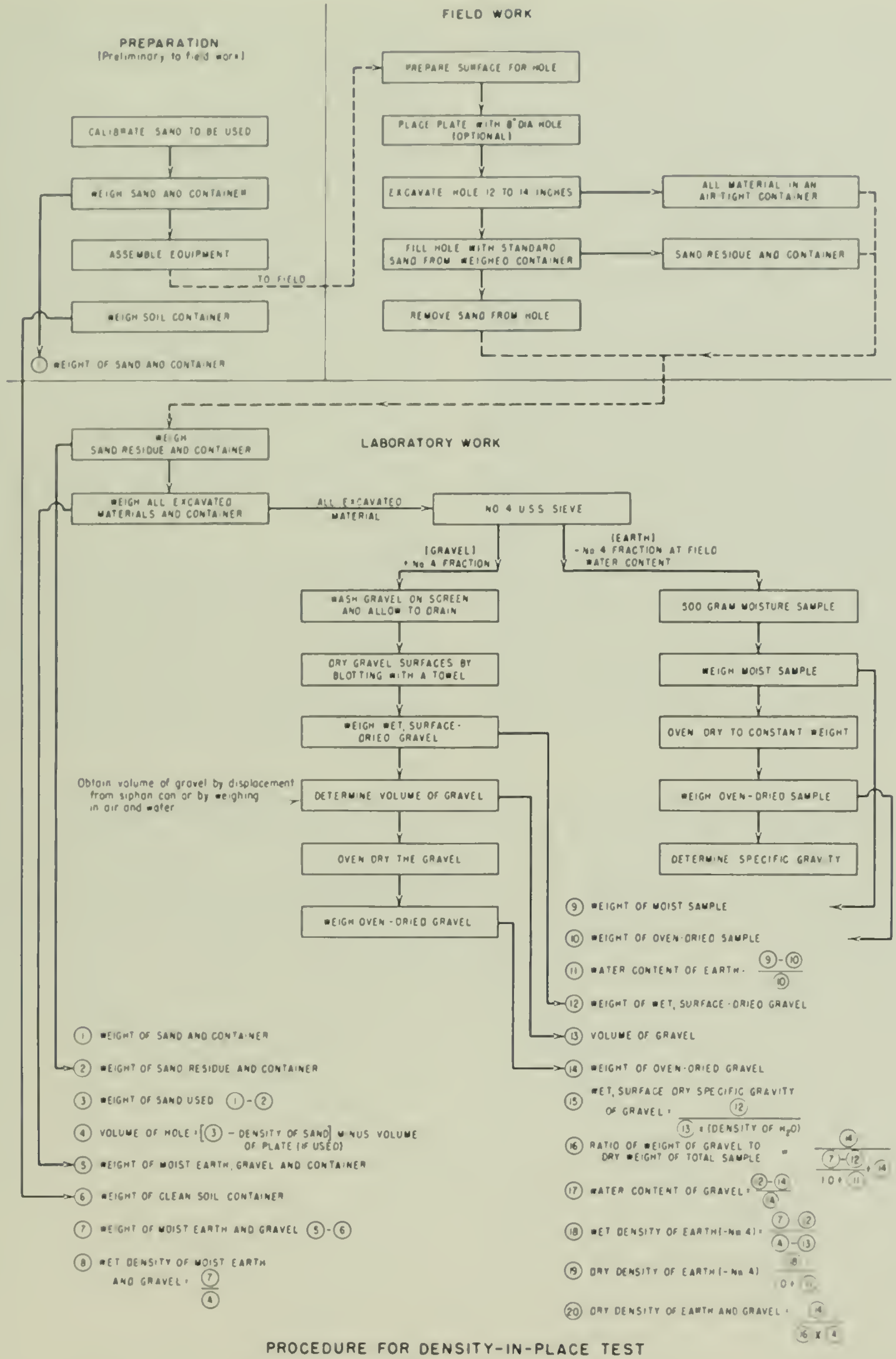


Figure 88. Procedure for density-in-place test.

about 1 cubic foot of material is extracted for a 3-foot depth, an accuracy of weighing to about 1 pound is satisfactory. A field scale of 150- or 200-pound capacity can be used.

This method is applicable only to relatively dry, cohesive soils. If the hole tends to cave, this method obviously fails and test pits must be used.

115. *Laboratory Test on Soils.*—(a) *Gradation.*—The gradation or grain size analysis of soils is done by a combination of sieving and wet mechanical analysis. A representative sample of the soil is dried, weighed, and screened on a United States standard No. 4 screen to remove the gravel which is then passed through a series of screens to determine the amount larger than 3 inches, 1½ inches, ¾ inch, ⅜ inch, and ¼ inch. An oven-dried sample of the minus No. 4 material is used for the remainder of the test. One-hundred grams of soil for sands (50 grams for silts and clays) are carefully weighed out and treated with 20 cc. of 0.5 normal sodium silicate solution and distilled water in order to separate the fine grains. After several hours the mixture is dispersed by thoroughly mixing in a malted milk type of machine, then transferred to a 1,000-ml. graduate. Distilled water is added to bring it to exactly 1,000 ml. and mixed.

The graduate containing the mixture is placed upon a table, and a stopwatch is started. A soil hydrometer is placed in the mixture and readings are made at 1, 4, 19, and 60 minutes; and also at 7 hours 15 minutes when expansive clays are involved. The hydrometer is of the Bouyoucos type, which is calibrated in grams per liter at 20° C., and its readings are corrected for the meniscus error (the top of the meniscus is read during the test), for difference in temperature from 20° C., and for the amount of deflocculating agent used. On completion of the 1-hour or the 7-hour 15-minute reading the mixture is washed on a No. 200 United States standard sieve and the retained fraction is dried and separated on the Nos. 8, 16, 30, 50, 100, and 200 standard sieves; 15 minutes of shaking in a power sieve shaker is usually done. The residue on each screen is weighed. Figure 89 is an example of a resulting gradation analysis curve.

(b) *Water Content.*—The water content of a soil is defined as the weight of water it contains divided by the weight of dry soil. The procedure involves weighing a sample of moist soil and its container and drying it in an oven at 110° C. to constant weight. The time required to attain constant weight varies for different soils, from a

FORM 7-1565

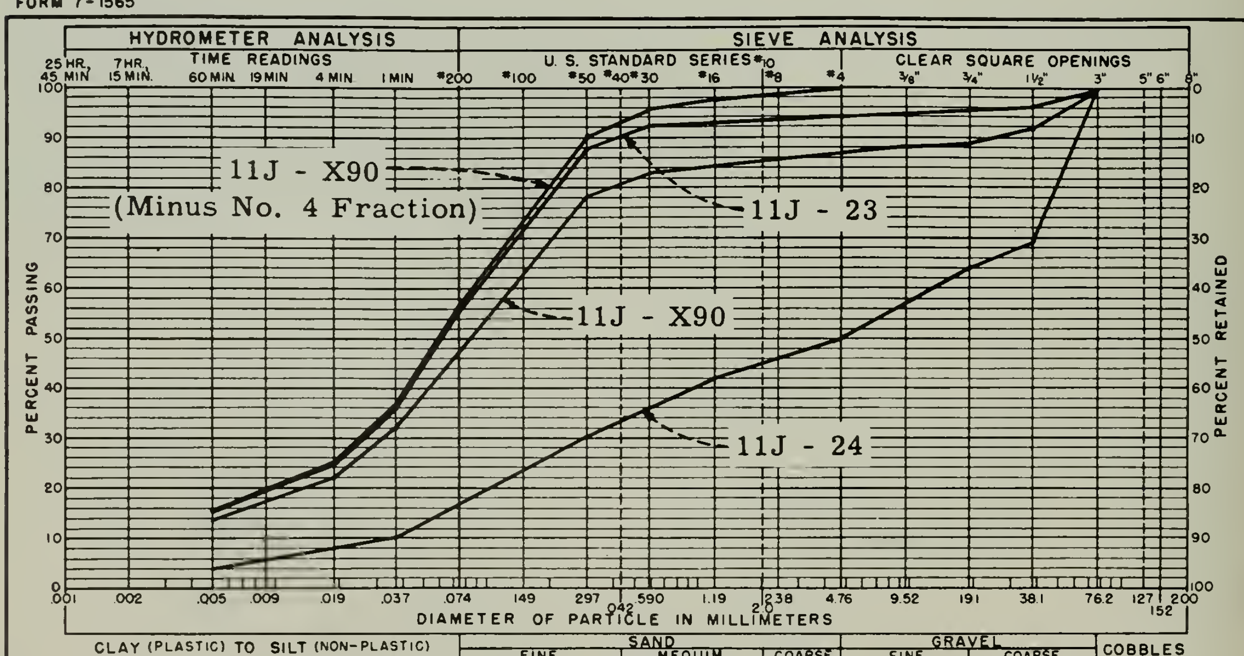


Figure 89. **Gradation analysis curves.**

few hours for sandy soils to several days for very fat clays. About 16 hours should be the minimum time used. The dried sample and container are placed in a desiccator to cool to room temperature prior to weighing. The water content is calculated as being the difference between the initial and final weights of soil and container, divided by the difference between the weight of the dry soil and container, and the weight of the container. In order to insure accuracy, the following sizes of water content samples are recommended:

Size of sample, grams	*Size of soil particles*
10	Minus No. 40.
200	Minus No. 4.
500	Minus ⅜-inch.
1,000	Minus ¾-inch.
2,000	Minus 1½-inch.
2,000 or more [1]	No. 4 to 3-inch gravel.

[1] The sample should be large enough to get a representative sample of the material up to the 3-inch size.

(c) *Atterberg Limits.* To obtain the liquid limit of a soil, the fraction passing the No. 40 sieve is mixed with water to a puttylike consistency and placed in a brass cup, as shown in figure 35. It is leveled off to a depth of 1 cm. and divided by a grooving tool, as shown in the figure. The crank is turned two rotations per second until the two sides of the sample come in contact at the bottom of the groove for a distance of one-half inch along the groove, and the number of blows is recorded. The water content of soil taken from this portion of the groove is determined. The test is repeated with added water or with less water until a result of 25 blows is bracketed; that is, test results above and below 25 blows are obtained. A "flow curve" is then plotted on a semilogarithmic graph with the number of blows on the logarithmic scale against the water content on the arithmetic scale. The water content corresponding to the 25-blow value is the liquid limit. Detailed test procedures are given in ASTM Designation D 423–54T.

The plastic limit is the lowest moisture content expressed as a percentage of the weight of oven-dried soil at which the soil can be rolled into threads one-eighth inch in diameter without the thread breaking into pieces. In order to determine it, about 15 grams of minus No. 40 fraction of a soil are mixed with enough water to obtain a plastic material and shaped into a ball. The soil is then rolled between the palm of the hand and a ground glass plate or absorbent paper to form the soil into a thread one-eighth inch in diameter. It is then reformed into a ball, kneaded and rolled out again. This procedure is continued until the soil crumbles when the thread becomes one-eighth inch and cannot be reformed. The water content determined for this condition is the plastic limit. Figure 36 shows the test for plastic limit. The plasticity index is the difference between the liquid limit and the plastic limit of a soil. Detailed test procedures are given in ASTM Designation 424–54T.

(d) *Specific Gravity.*—Specific gravity is defined as the ratio of the weight in air of a given volume of material to the weight in air of an equal volume of distilled water at a stated temperature. The minus No. 4 fraction of soil is commonly tested for specific gravity by the flask method. In this method a 500-ml., long-necked flask is calibrated for volume at several temperatures. One hundred grams of oven-dried minus No. 4 material is washed into the calibrated flask with distilled water. With the water level well below the neck of the flask, a vacuum is applied to the mixture which causes boiling of entrapped air from the mixture. When the air has been virtually exhausted, distilled water is added to bring the volume to exactly the calibrated volume of the flask, and the vacuum is applied again. When all the air has been removed, the flask and its contents are weighed, and the temperature of the mixture is determined. The volume of the 100 grams of dried soil is determined from the data obtained, and the specific gravity of the soil is then computed.

To determine the specific gravity of gravel and cobbles, the material is immersed in water for a period of 24 hours and then blotted with a towel. This is the saturated surface-dry condition. It is then weighed and carefully placed in a filled siphon can from which the volume of water it displaces is measured. The bulk specific gravity on a saturated surface-dry basis is the weight of the sample divided by the volume of water displaced. The bulk specific gravity on an oven-dry basis is the oven-dry weight of the material divided by the volume displaced by the saturated surface-dry material. See section 116(a) for another method of specific gravity determination.

(e) *Proctor Compaction.*—The Proctor maximum dry density of a soil is the greatest dry unit weight obtainable by the method to be described. The

optimum water content of the soil is the water content at this condition. For this test, water is added to about 35 pounds of the minus No. 4 fraction of the soil until its consistency is such that it barely adheres when squeezed firmly in the hand. A sample of the soil is compacted in a $\frac{1}{20}$-cubic-foot Proctor mold (with collar attached) in 3 equal layers by 25 uniformly distributed blows per layer of a tamping rod weighing 5.5 pounds dropped freely 18 inches above the layer. The third compacted layer should extend slightly into the collar section. The collar is then removed, and the soil is trimmed to the top of the mold with a straightedge trimmer. The soil and mold are then weighed. The water content of the compacted specimen is determined from a sample taken near the center. This procedure is repeated at least five times using new soil for each specimen and increasing the water added until the resulting compacted wet weight decreases.

The Proctor mold used by the Bureau of Reclamation is one-twentieth cubic foot in volume, and the foregoing procedure with that mold results in a compactive effort of 12,375 foot-pounds per cubic foot of soil. The ASTM Designation D 698–42T and the standard AASHO methods use the same compactive effort, 12,375 foot-pounds per cubic foot, and identical procedures, except that a $\frac{1}{30}$-cubic-foot cylinder is used and the free drop is 12 inches instead of 18 inches.

The penetration resistance of the compacted soil for points along the compaction curve, as shown in figure 90, can be obtained by forcing the Proctor needle into each compacted specimen and determining the penetration resistance in pounds per square inch. This method has been extensively used for moisture control of compacted fills. However, the rapid method of compaction control described in appendix E is believed to be a more accurate method which should supplant the Proctor needle for that purpose.

(f) *Relative Density.*—Relative density is defined as the state of compactness of a soil with respect to the loosest and densest states at which it can be placed by specific laboratory procedures. This test is applicable to cohesionless materials which do not have well-defined Proctor curves. The minimum density is obtained by carefully placing dried soil in a container of known size, usually ½ to 1 cubic foot. About 1 inch free fall is permitted for sands; gravel up to 3 inches in size is placed with a scoop. The excess soil is carefully trimmed level to the top and the full container is weighed.

To obtain the maximum density, the soil is thoroughly wetted and placed slowly into the container with an attached vibrator operating. After the container is filled, the vibrator is operated for at least 1 minute. The material in the container is then emptied into a pan, dried, and weighed. The relative density is defined by the formula:

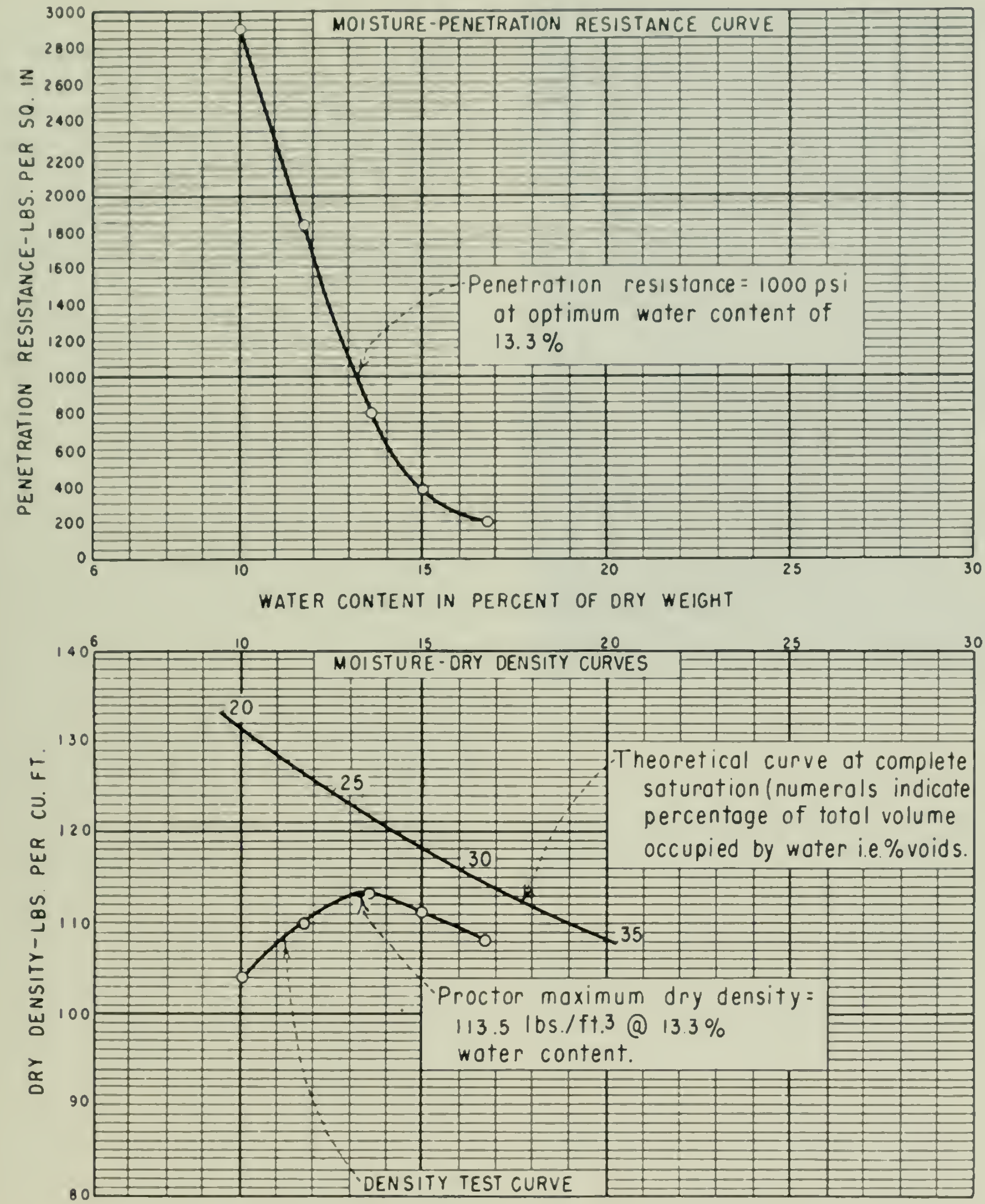
$$D_d=\frac{e_{max}-e}{e_{max}-e_{min}}=\frac{\gamma_{D_{max}}(\gamma_D-\gamma_{D_{min}})}{\gamma_D(\gamma_{D_{max}}-\gamma_{D_{min}})} \qquad (5)$$

It is usually expressed as a percentage. Figure 91 shows a maximum density test in progress, using a 0.5-cubic-foot container.

116. *Laboratory Tests on Riprap and Concrete Aggregate.*—(a) *Specific Gravity and Absorption.*—The specific gravity of sand for concrete aggregate can be determined on a saturated surface-dry sample in a manner similar to that given for soil in section 115(d). The specific gravity of coarse aggregates and riprap (crushed to 1½-inch maximum size) is determined by saturating the material for 24 hours in water at a temperature of 59° to 77° F., blotting with a towel, and weighing. After weighing, the material is placed in a wire basket and is weighed again in water. The sample is then dried to a constant weight in an oven, cooled to room temperature, and weighed again. If A is the weight in grams of the oven-dried sample in air, B the weight in grams of the saturated surface-dried sample in air, and C the weight in grams of the sample in water, then the specific gravity, on a dry basis, equals $\frac{A}{B-C}$, the specific gravity on a saturated surface-dry basis equals $\frac{B}{B-C}$, and the absorption equals $\frac{B-A}{A}$ on a dry basis and $\frac{B-A}{B}$ on a saturated surface-dry basis.

Absorption is usually expressed as a percentage. ASTM Designation C 128–42 describes detailed procedures for these tests.

(b) *Abrasion.*—This test determines the abrasion resistance of crushed rock and natural and



COMPACTION		SOIL PROPERTIES	
25	BLOWS PER LAYER	2.66	SPECIFIC GRAVITY
3	LAYERS	SM	SOIL CLASSIFICATION
5.5	LB. HAMMER	11.2	% LARGER THAN TESTED
18	IN. DROP	113.5	MAX. DRY DENSITY (P.C.F.)
1/20	C.F. CYLINDER	13.3	OPT WATER CONTENT (%)
		1000	PEN. RES. AT OPT MOIST. (P.S.I.)

Figure 90. Proctor compaction test curves.

Figure 91. **A maximum density test in progress using the 0.5-cubic-foot container.**

crushed gravel. The Los Angeles abrasion machine, which consists of a hollow steel cylinder closed at both ends, having a diameter of 28 inches and a length of 20 inches, is used.

The abrasive charge consists of cast-iron or steel spheres approximately 1⅞ inches in diameter. Twelve spheres are used for an A grading (maximum size of particle 1½ inches), 11 for a B grading (¾ inch maximum), and 8 for a C grading (⅜ inch maximum).

The test sample of 5,000 grams and the proper abrasive charge are placed in the Los Angeles abrasion testing machine, and the machine is rotated for 100 revolutions at a speed of from 30 to 33 revolutions per minute. The material is then removed from the machine, screened on a No. 12 screen, and the material retained on the screen weighed. The entire sample including the dust of abrasion is returned to the testing machine, the machine is rotated an additional 400 revolutions, and the screening and weighing are repeated. The differences between the original weight of the test sample and the weights of the material retained on the screen at 100 revolutions and at 500 revolutions are expressed as percentages of the original weight of the test sample. These values are reported as percentages of wear. ASTM Designation C 131-55 describes detailed procedures for this test.

(c) *Soundness.*—The most commonly used soundness test is the sodium sulfate test. The results of this test are used as an indication of the ability of aggregate and riprap to resist weathering. A carefully prepared saturated solution of sodium sulfate is kept at a temperature of 73.4° F. (23° C.). After washing and drying in an oven, the material to be tested is screened to provide a specified gradation, usually from 1½ inches to the No. 50 sieve size. Specified weights of the various fractions of the material are placed in separate containers resistant to the action of the solution, and sufficient sodium sulfate solution is poured into the containers to cover the samples. The material is permitted to soak for 18 hours, during which the temperature is maintained at 73.4° F. (23° C.).

After the 18-hour immersion period, the samples are removed from the solution and dried to constant weight (about 4 hours) at a temperature of 221° to 230° F. (105° to 110° C.). After drying, the sample fractions are cooled to room temperature and the process is repeated. At the end of five cycles, the test sample is inspected and records made of observation. Each fraction is then washed thoroughly to remove the sodium sulfate from the material, dried, and cooled. Each fraction is screened and the quantities of material retained are weighed. The weighted average loss for each fraction is computed and reported. ASTM Designation C 88-55T describes detailed procedure for this test.

J. BIBLIOGRAPHY

117. *Bibliography.*

[1] Howell, J. V., and Levorsen, A. I., "Directory of Geological Material in North America," American Geological Institute, second edition, 1957, 208 pp.

[2] "Soil Survey Manual," U.S. Department of Agriculture, Handbook No. 18, 1951, p. 209.

[3] ———, p. 473.

[4] "ASTM Manual on Quality Control of Materials," American Society for Testing Materials, Special Publication 15-C, pt. 2, January 1951, pp. 41–51.

[5] "Geology and Its Military Applications," U.S. Department of the Army, Technical Manual TM 5-545, 1952, pp. 3–32.

[6] "Earth Manual," U.S. Department of the Interior, Bureau of Reclamation, tentative edition, February 1957, designation E 2, p. 4.

[7] ———, designations E 5, E 6, E 8, E 9, E 10, and E 11.

[8] "Concrete Manual," U.S. Department of the Interior, Bureau of Reclamation, sixth edition, revised 1956, designations 10, 19, and 21.

[9] "Cementitious Materials, Refractories, Glass, Ceramics, Thermal Insulation, Concrete, Mortar, Stone, Building Units, Road Materials, Waterproofing, Roofing, Soils," American Society for Testing Materials, 1955 Book of ASTM Standards, pt. 3, 1955, 2,017 pp.

Earthfill Dams

H. G. ARTHUR [1]

A. INTRODUCTION

118. *Origin and Development.*—Earthfill dams for the storage of water for irrigation, as attested both by history and the surviving remnants of ancient structures, have been used since the early days of civilization. Some of the structures built in antiquity were of considerable size. One earthfill dam 11 miles long, 70 feet high, and containing about 17 million cubic yards of embankment was completed in Ceylon in the year 504 B.C. [1].[2] Today, as in the past, the earthfill dam continues to be the most common type of small dam, principally because its construction involves utilization of materials in their natural state with a minimum of processing.

Until modern times all earthfill dams were designed by empirical methods, and engineering literature is replete with accounts of failures [2]. These failures compelled the realization that empirical methods must be replaced by rational engineering procedures in both design and construction. One of the first to suggest that the slopes for earthfill dams be selected on that basis was Bassell in 1907 [3]. However, little progress was made on the development of rational design procedures until the 1930's. The rapid advancement of the science of soil mechanics since that time has resulted in the development of greatly improved procedures for the design of earthfill dams. These procedures include (1) thorough preconstruction investigations of foundation conditions and materials of construction; (2) application of engineering skill and technique to design; and (3) carefully planned and controlled methods of construction.

[1] Engineer, Earth Dams Section, Bureau of Reclamation.

[2] Numbers in brackets refer to items in the bibliography, sec. 143.

As a result, earthfill dams are now (1958) being constructed to heights exceeding 500 feet above their foundations; and hundreds of large rolled earthfill dams have been constructed in the past 20 years without a single recorded failure. Failures of small earthfill dams, however, continue to be commonplace. Though some of these failures likely are a result of improper design, many of them are caused by lack of care in construction. Proper construction methods include adequate foundation preparation and the placement of materials in the dam embankment with the necessary degree of compaction under an established procedure of testing and control.

The design of an earthfill dam must be realistic. It should reflect the actual foundation conditions at the site and the available materials for embankment construction, and not merely be patterned after a successful design used at a site with dissimilar conditions.

119. *Scope of Discussion.*—This discussion is limited to design procedures for small earthfill dams which are of the rolled-fill type of construction, as defined in section 120. This type of construction is now being used almost entirely for the construction of small dams to the exclusion of semihydraulic and hydraulic fills.

For the purpose of this discussion a "small" dam is one whose maximum height above the lowest point in the original streambed does not exceed 50 feet, and whose volume is not of such magnitude that significant economies can be obtained by utilizing the more precise methods of designs usually reserved for large dams. A low dam cannot be considered small if its volume exceeds say, 1 million cubic yards. Figures 92 and

Figure 92. **Upstream face of dam and fishscreened inlet structure at Crane Prairie Dam on the Deschutes River in Oregon. Deschutes 93.**

19 show typical small dams constructed by the Bureau of Reclamation. Crane Prairie Dam was completed in 1940. It has a height of 31 feet and contains 29,700 cubic yards. Crescent Lake Dam was completed in 1956. It has a height of 22 feet and contains 16,800 cubic yards. Maximum sections of these dams are shown in figures 135 and 136, respectively.

Figures 93 and 94 show dams constructed by the Bureau of Reclamation which are at the upper limit of height for the scope of this text; in fact, Fruitgrowers Dam is slightly above the height limit. It has a maximum height of 55 feet and a volume of 135,500 cubic yards, but is included herein as a matter of interest. Irrigation at this site dates back to 1898; the dam shown here was constructed in 1939 downstream from the original structure, which was breached in June 1937 to forestall failure. A maximum section of Fruitgrowers Dam is shown in figure 139. Shadow Mountain Dam is a 50-foot-high structure containing 168,000 cubic yards of embankment, completed in 1946. Its maximum section is shown in figure 150.

The design procedures given in this text are not sufficiently detailed to permit their sole use for the design of small dams where exceedingly soft or exceedingly pervious foundations are involved, nor where the nature of the only soil available for construction of the embankment is unusual. In this category are soils of high plasticity, low maximum density, or very high natural water content which cannot be reduced by drainage. These conditions require the services of an engineer specializing in earthfill dam design to direct the investigations, to determine the laboratory testing program, to interpret the laboratory test results, and to supervise the preparation of the design and specifications.

120. *Selection of Type of Earthfill Dam.*—(a)

Figure 93. **Fruitgrowers Dam, an earthfill storage dam at an offstream location in Colorado. Washington Office 39.**

General.— The selection of type of dam (earthfill, rockfill, concrete gravity, or combination of these types is discussed in chapter III. When this procedure leads to the selection of an earthfill dam, a further decision must be made as to the type of earthfill dam.

The scope of this text includes only the rolled-fill type of earthfill dam. In this type the major portion of the embankment is constructed in successive, mechanically compacted layers. The material from borrow pits and that which is suitable from excavations for other structures is delivered to the embankment, usually by trucks or scrapers. It is then spread by motor-patrols or bulldozers and sprinkled, if necessary, to form layers of limited thickness having the proper moisture content, which are then thoroughly compacted and bonded with the preceding layer by means of power-operated rollers of proper design and weight. Rolled-fill dams are of three types; namely, diaphragm, homogeneous, and zoned.

(b) *Diaphragm Type.*—In this type of section the bulk of the embankment is constructed of pervious material (sand, gravel, or rock) and a thin diaphragm of impermeable material is provided to form the water barrier. The position of this diaphragm may vary from a blanket on the upstream face to a central vertical core. The diaphragm may consist of earth, portland cement concrete, bituminous concrete, or other material. If the blanket or core is of earth, it is considered to be a "diaphragm" if its horizontal thickness at any elevation is less than 10 feet or less than the height of embankment above any corresponding elevation in the dam. If the impervious earth zone equals or exceeds these thicknesses, the design is considered to be of the "zoned embankment" type.

If the bulk of material comprising the diaphragm type dam is rock, the dam is classified as a rockfill dam. The design of rockfill dams is discussed in chapter VI.

Although successful dams have been constructed with internal or "buried" diaphragms, this type of construction is not recommended for structures within the scope of this text. The construction of an internal diaphragm of earth with the necessary filters requires a higher degree of precision and closer control than it is feasible to obtain for small dams. Internal diaphragms of rigid material such as concrete also have the disadvantage of not being readily available for

Figure 94. Shadow Mountain Dam, an earthfill structure on the Colorado River in Colorado, constructed as part of a large transmountain diversion scheme. SM–175–CBT.

inspection or emergency repair if they are ruptured due to settlement of the dam or its foundation.

An earth blanket on the upstream slope of an otherwise pervious dam also is not recommended because of the expense and the difficulty of construction of suitable filters. Furthermore, the earth blanket must be protected from erosion by wave action, so it is buried and not readily available for inspection or repair. If the supply of impervious soil is so limited that the "zoned embankment" type of dam cannot be constructed, a diaphragm of manufactured material placed on the upstream slope of an otherwise pervious embankment is recommended for small dams. The design of suitable impervious pavings is discussed in chapter VI.

(c) *Homogeneous Type.*—A purely homogeneous type of dam is composed of a single kind of material (exclusive of the slope protection). The material comprising the dam must be sufficiently impervious to provide an adequate water barrier and the slopes must be relatively flat for stability. To avoid sloughing, the upstream slope must be relatively flat if rapid drawdown of the reservoir after long-term storage is anticipated. The downstream slope must be relatively flat to provide a slope sufficiently stable to resist sloughing when saturated to a high level. For a completely homogeneous section it is inevitable that seepage will emerge on the downstream slope regardless of its flatness and the impermeability of the soil, if the reservoir level is maintained for a sufficiently long time. The downstream slope eventually will be affected by seepage to a height of roughly one-third the depth of the reservoir pool [4], as shown in figure 95.

Although formerly very common in the design of small dams, the purely homogeneous section has been replaced by a modified homogeneous section in which small amounts of carefully placed pervious materials control the action of seepage so as

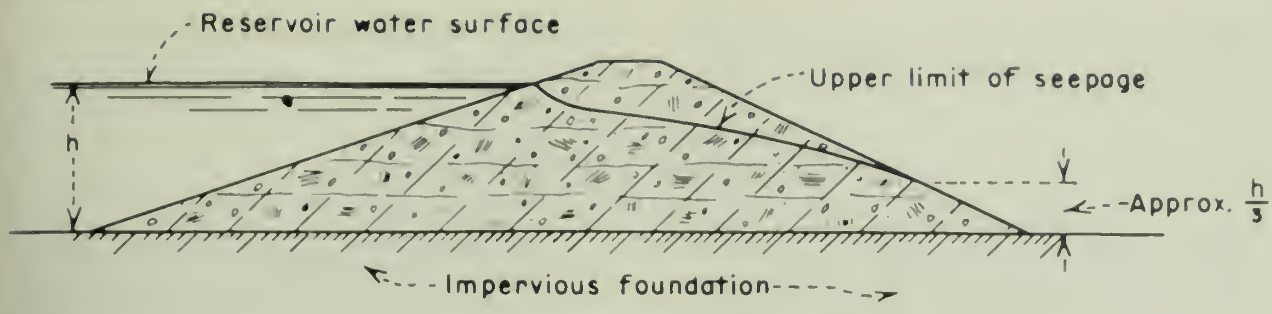


Figure 95. Completely homogeneous dam.

to permit much steeper slopes. The effect of drainage at the downstream toe of the embankment is shown in figure 96.

Rock toes of appreciable size may be provided for drainage, as shown in figure 96(A), or, if suitably graded materials are available, the filter drain shown in figure 96(B) may be used. Another method of providing drainage which has been used is the installation of pipe drains. These are recommended for small dams only when used in conjunction with filter drains or pervious zones. Reliance should not be placed solely upon pipe drains because of the possibility of the pipe becoming clogged as the result of improper filters, root growth, or deterioration.

Because the modification of the homogeneous type of section by means of drainage furnishes a greatly improved design, the fully homogeneous section should not be used for storage dams; drainage should always be provided when a reservoir pool will be maintained for an appreciable length of time. A homogeneous (or modified-homogeneous) type of dam is applicable in localities where readily available soils show little variation in permeability and soils of contrasting permeabilities are available only in minor amounts or at considerably greater cost.

(d) *Zoned Embankment Type.*—The most common type of a rolled earthfill dam section is that in which a central impervious core is flanked by zones of materials considerably more pervious. The pervious zones enclose, support, and protect the impervious core; the upstream pervious zone affords stability against rapid drawdown; and the downstream pervious zone acts as a drain to control the line of seepage. For most effective control of through seepage and drawdown seepage, the section should show, to the extent feasible, a progressive increase in permeability from the center out toward each slope.

The pervious zones may consist of sand, gravel, cobbles, or rock, or mixtures of these materials. For purposes of this text, the dam is considered to be a zoned embankment if the horizontal width of the impervious zone at any elevation equals or exceeds the height of embankment above that elevation in the dam, and is not less than 10 feet. The maximum width of the impervious zone will be controlled by stability and seepage criteria and by the availability of material. A dam with an

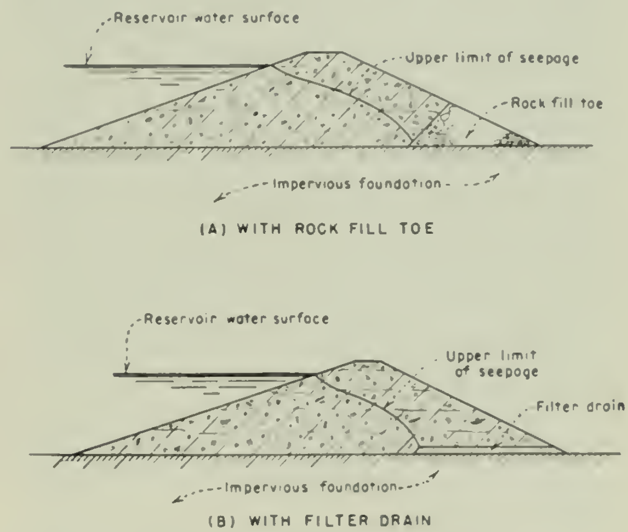


Figure 96. Modified homogeneous dam.

impervious core of moderate width composed of strong material and with pervious outer shells may have relatively steep outer slopes, limited only by the strength of the foundation, the stability of the embankment itself, and maintenance considerations. Conditions that tend to increase stability may be decisive in the choice of a section even if a longer haul is necessary to obtain required embankment materials.

It may be said that if a variety of soils are readily

available, the choice of type of earthfill dam should always be the zoned embankment type because its inherent advantages will lead to economies in cost of construction.

B. DESIGN PRINCIPLES

121. ***Design Data.***—The data required for the design of an earthfill dam are discussed in chapter I, and chapter IV describes the investigation of foundations and sources of construction materials. The required detail and accuracy of the data will be governed by the nature of the project and the immediate purpose of the design; that is, whether the design is intended as a basis for a cost estimate to determine project feasibility, whether the design is for the purpose of construction, or whether some intermediate purpose is to be served. The extent of investigations of foundations and sources of construction material will also be governed by the complexity of the situation.

122. ***Criteria for Design.***—The basic principle of design is to produce a satisfactory functional structure at a minimum total cost. Consideration must be given to maintenance requirements so that economies achieved in the initial cost of construction will not result in excessive maintenance costs. The latter costs will vary with the provisions of upstream and downstream slope protection, drainage features, and the type of appurtenant structures and mechanical equipment. For minimum cost, the dam must be designed for maximum utilization of the most economical materials available, including material which must be excavated for its foundations and for appurtenant structures.

An earthfill dam must be safe and stable during all phases of construction and operation of the reservoir. To accomplish this, the following criteria must be met:

(1) The embankment must be safe against overtopping during occurrence of the inflow design flood by the provision of sufficient spillway and outlet works capacity.

(2) The slopes of the embankment must be stable during construction and under all conditions of reservoir operation, including rapid drawdown of the reservoir in the case of storage dams.

(3) The embankment must be designed so as not to impose excessive stresses upon the foundation.

(4) Seepage flow through the embankment, foundation, and abutments must be controlled so that no internal erosion takes place and so there is no sloughing in the area where the seepage emerges. The amount of water lost through seepage must be controlled so that it does not interfere with planned project functions.

(5) The embankment must be safe against overtopping by wave action.

(6) The upstream slope must be protected against erosion by wave action, and the crest and downstream slope must be protected against erosion due to wind and rain.

An earthfill dam designed to meet the above criteria will prove permanently safe provided proper construction methods and control are achieved. The design procedure to meet the requirements of criterion (1) above is discussed in chapter VIII. Methods for satisfying other criteria for earthfill dams subject to the limitations in scope described in section 119 will be discussed herein. The applicability of the procedures to a specific case will depend upon the purpose of the design, the size and importance of the structure, and the complexity of the problems.

C. FOUNDATION DESIGN

123. ***General.***—The term "foundation" as used herein includes both the valley floor and the abutments. The essential requirements of a foundation for an earthfill dam are that it provide stable support for the embankment under all conditions of saturation and loading and that it provide sufficient resistance to seepage to prevent excessive loss of water.

Although the foundation is not actually designed, certain provisions for treatment are made in designs to assure that the essential requirements will be met. No two foundations are exactly alike; each foundation presents its own separate and distinct problems requiring corresponding special treatment and preparation. Various methods of stabilization of weak foundations, reduction of seepage in permeable foundations, and types and locations of devices for the interception of underseepage must depend upon, and be adapted to local conditions.

Theoretical solutions based on principles of soil mechanics can be made for problems involving pervious or weak foundations. Most of these solutions are relatively complex and they may be relied upon only to the degree that the actual permeabilities in various directions or the strength of the foundation can be determined by expensive, detailed field and laboratory testing. Ordinarily, extensive exploration of this nature and complex theoretical designs are not required for the small dams which are discussed in this text. For these structures it is usually more economical to design foundations on the basis of judgment, deliberately striving for substantial factors of safety. The savings in construction cost that can be achieved by more precise design ordinarily do not warrant the cost of additional exploration, testing, and engineering involved. There are foundations, however, where conditions are unusual to the extent that judgment cannot be relied upon to produce a design with an adequate factor of safety. Such conditions require the services of an engineer specializing in the field of earthfill dam design and are beyond the scope of this text.

Because different types of treatment are appropriate for different conditions, foundations are grouped into three main classes according to their predominant characteristics:

Foundations of rock.

Foundations of coarse-grained material (sand and gravel).

Foundations of fine-grained material (silt and clay).

It is realized that foundations which originate from various sources such as river alluvium, glacial outwash, talus, and other processes of erosion, disintegration, and deposition are characterized by infinite variations in the combinations, structural arrangement, and physical characteristics of their constituent materials. The deposits may be roughly stratified, containing layers of clay, silt, fine sand and gravel; or they may consist of lenticular masses of the same material without any regularity of occurrence and of varying extent and thickness. In spite of this, the character of the foundation, as revealed by exploration, usually can be safely generalized for the design of small dams to fit one of the classes given above, and the nature of the problem requiring treatment will be evident. Ordinarily, pervious foundations present no difficulties in the matter of settlement or stability for a small dam; conversely, problems of seepage are not associated with weak foundations subject to settlement or displacement.

The special treatments required for the different types of foundations listed above are discussed in this chapter. If the foundation material is impervious and comparable to the compacted embankment material in structural characteristics, little foundation treatment will be required. *The minimum treatment for any foundation* is stripping of the foundation area to remove sod, topsoil with high content of organic matter, and other unsuitable material that can be disposed of by open excavation. In many cases where the overburden is comparatively shallow, the entire foundation is stripped to bedrock. *In all soil foundations in which a cutoff trench or partial cutoff trench (see sec. 126) is not used*, a "key" trench should be provided to bond the impervious zone of the embankment to the foundation. The top several feet of the soil foundation, which invariably lacks the density of the underlying soil due to frost action, surface runoff, wind, or other cause, should be penetrated by the key trench. A bottom width of 20 feet for the key trench is usually sufficient.

The foundation at any one particular site usually consists of a combination of the various classes. For example, the stream portion often is a sand-gravel foundation, while the abutments are rock which is exposed on the steep slopes and mantled by deep deposits of clay or silt on the gentle slopes. Therefore, the design of any one dam may involve a variety of foundation design problems.

124. *Rock Foundations.* Foundations of rock, including hard shale, do not present any problem of bearing strength for a small earthfill dam. The principal considerations are dangerous, erosive leakage and the excessive loss of water through joints, fissures, crevices, permeable strata, and

along fault planes. Ordinarily, the design and estimate for a storage dam should provide for the injection of grout under pressure to seal seams, joints, and other openings in the bedrock to a depth equal to the reservoir head above the surface of the bedrock. Grouting is usually done with neat cement and water starting with a ratio of 1 : 5. If considerable "take" in any one hole is experienced, the grout mixture is thickened progressively until a ratio of 1 : 1 is obtained. Admixtures, such as sand or clay, are added if large voids are encountered. For small dams, a single line of grout holes is sufficient. Grouting of rock foundations ordinarily will not be required for detention dams, very small diversion and storage dams, or where preconstruction exploration has demonstrated that there are no openings in the bedrock.

Very often the bedrock is found to be badly jointed or broken for some depth below its surface. In such instances, a concrete grout cap may be needed to facilitate grouting. The grout cap is usually a concrete-filled trench excavated a minimum of 3 feet and a maximum of 8 feet in the bedrock, depending on conditions. The trench is usually made a minimum width of 3 feet to facilitate construction. The grout cap performs several functions: (1) It provides anchorage for the pipe nipple to which the grout pump is connected; (2) it cuts off seepage in the upper portion of the bedrock which cannot be successfully grouted; and (3) it provides weight so that higher grouting pressures may be used at shallow depths. Excavation for the grout cap must be carefully performed so that the rock is not shattered.

Typical specifications for the performance of grouting and for the excavation for the grout cap are included in appendix G. However, if an extensive grouting program is required in the construction of a dam, consulting advice should be obtained from an engineer experienced in this type of work. For additional information on the subject of grouting, including the experiences of the Bureau of Reclamation, the U.S. Corps of Engineers, and the Tennessee Valley Authority, refer to the American Society of Civil Engineers' papers which comprise the "Symposium on Cement and Clay Grouting of Foundations" [5].

Concrete cutoff walls were formerly provided for even small dams to intercept seepage along the contact of the embankment with the rock foundation. These walls are expensive and are ordinarily not required for small earthfill dams of the type recommended in this text, provided that care is taken to obtain an intimate contact between the impervious portion of the dam and the abutments. In unusual cases where the bedrock is very smooth, a cutoff wall may be warranted. All loose and overhanging rock must be removed from the abutments and the slopes should be flattened to 1 to 1. Where this is not practicable, a short section of cutoff concrete wall 5 feet high should be provided.

If the bedrock is a shale which air slakes, it may be necessary to excavate several feet into bedrock to remove the surface disintegration just prior to placement of the embankment; otherwise excavation into the bedrock (other than for a grout cap) is seldom required. A sample specifications for construction on a shale foundation subject to slaking is included in appendix G.

In most instances, bedrock is mantled by overburden of various types and thicknesses. The foundation design then depends on the nature and depth of the overburden, as described in succeeding sections. The above discussion is applicable not only to exposed rock foundations, but also to bedrock which is reached by trenching through the overburden.

125. *Characteristics of Sand and Gravel Foundations.*—(a) *General.*—Often the foundations for dams consist of recent alluvial deposits composed of relatively pervious sands and gravels overlying impervious geological formations. The pervious materials may range from fine sand to openwork gravels, but more often they consist of stratified heterogeneous mixtures.

Two basic problems are found in pervious foundations; one pertains to the amount of underseepage, and the other is concerned with the forces exerted by the seepage. The type and extent of treatment justified to decrease the amount of seepage will be determined by the purpose of the dam, the streamflow yield in relation to the reservoir conservation capacity, and the necessity for making constant reservoir releases to serve senior water rights or to maintain a live stream for fish, etc. Loss of water through underseepage may be of economic concern for a storage dam but of little consequence for a detention dam. Economic studies of the value of the water and the cost of limiting the amount of underseepage are required in some instances to

determine the extent of treatment. On the other hand, adequate measures must be taken to insure the safety of the dam against failure due to piping, regardless of the economic value of the seepage.

A special problem may exist in foundations which are inherently unstable because they consist of clean saturated sand (usually fine and uniform) of very low density. The potential instability is caused by the loose structure of the sand which is subject to collapse under the action of a dynamic load. Although the loose sand may support small static loads through point-to-point contact of the sand grains, a vibration or shock may cause readjustment of the grains into a more dense structure with a squeezing out of water from the pores. Since drainage cannot take place instantaneously, part of the static load formerly carried by the sand grains is transferred temporarily to the water, and the foundation behaves as a liquid.

Foundations consisting of cohesionless sand of low density are suspect, and special investigations should be made to determine required remedial treatment if the relative density of the foundation is less than 50 percent. The approximate magnitude of the relative density of a cohesionless sand foundation can be determined from the results of standard penetration tests described in section 103. The number of blows per foot is related to the relative density, but is affected by the depth of the test and to some extent by the location of the water table. The following tabulation gives average standard penetration resistance values for 50 percent relative density irrespective of the water table, based on research by the Bureau of Reclamation [6].

Effective overburden pressure in p.s.i. (based on submerged unit weight)	*Number of blows per foot*
0	4
20	12
40	17

Special studies in triaxial shear on undisturbed samples may be required for foundations of cohesionless sand that are indicated to be below 50 percent relative density. Such studies are beyond the scope of this text, and the advice of specialists in dam design should be obtained.

(b) *Amount of Underseepage.* To estimate the volume of underseepage which may be expected, it is necessary to determine the coefficient of permeability of the pervious foundation. This coefficient is a function of the size and gradation of the coarse particles, the amount of fines, and the density of the mixture. Three general methods are used in the determination of the coefficient of permeability of foundations: (1) Pumping-out tests in which water is pumped from a well at a constant rate and the drawdown of the water table observed in wells placed on radial lines at various distances from the pumped well; (2) tests conducted by observation of the velocity of flow as measured by the rate of travel of a dye or electrolyte from the point of injection to an observation well; and (3) pumping-in tests in which water is pumped into a drill hole or test pit and the rate of seepage observed under a given head.

The pumping-out tests are relatively high in cost and the results are difficult to interpret. The rate-of-travel method is subject to the same limitations. The pumping-in tests are economical for use in design of small dams because they can be accomplished in conjunction with the usual exploratory drilling; however, the results can be considered as approximations only. Another advantage to the pumping-in tests in drill holes is that the permeability of various layers may be tested rather than only the overall permeability. Pumping-in tests in drill holes are discussed in chapter IV.

Upon determination of the coefficient of permeability of the foundation, a rough approximation of the amount of underseepage may be made by use of the Darcy formula:

$$Q=kiA \tag{1}$$

where:

Q=discharge in given unit of time,

k=coefficient of permeability for the foundation, i.e., discharge through a unit area at unit hydraulic gradient,

i=hydraulic gradient$=\frac{h}{L}$

$=\frac{\text{difference in head}}{\text{length of path}}$, and

A=gross area of foundation through which flow takes place.

The underseepage for the example shown in figure 97 is as follows:

k=25,000 feet per year

$$=\frac{25,000}{60\times60\times24\times365}$$

=0.00079 foot per second

h=elevation 210—elevation 175=35 feet

L=165 feet

$$i=\frac{h}{L}=\frac{35}{165}=0.212$$

Depth of foundation,

d=elevation 170—elevation 100=70 feet.

For a width of 1 foot, A=70 x 1=70 square feet. Q per foot of width=0.00079 x 0.212 x 70=0.012 second-feet.

For a foundation width of 100 feet, Q=1.2 second-feet; for a foundation width of 1,000 feet, Q=12 second-feet.

The accuracy of the amount of underseepage as determined by the Darcy formula depends on the homogeneity of the foundation and the accuracy with which the coefficient of permeability is determined. The results should be considered as an indication only of the order of magnitude of seepage in evaluation of water loss from a project use viewpoint.

If the foundation is stratified (as is usually the case), the vertical permeability will be much less than the horizontal permeability, and permeable layers at depth will not be fully effective in transmitting underseepage. The quantity of seepage as determined by equation (1) will be liberal if an average coefficient of permeability of the various layers, obtained by weighting each coefficient by the thickness of the layer, is used in the computations.

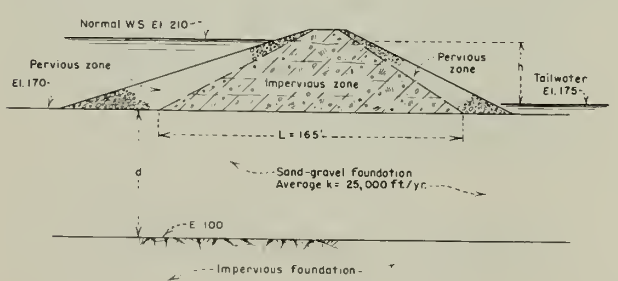


Figure 97. **Computation of seepage by Darcy's formula.**

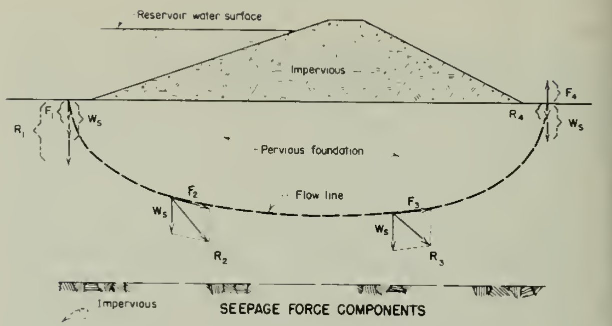


Figure 98. **Seepage force components.**

(c) *Seepage Forces.*—The flow of water through a pervious foundation produces seepage forces as a result of the friction between the percolating water and the walls of the pores of the soil through which it flows, similar to the friction developed by the flow of water through a pipe. Figure 98 shows the path of one filament of seepage flow through a pervious foundation of a dam. The water percolating downward at the upstream toe of the dam increases the submerged weight, W_s, of the soil by the initial seepage force, F_1, with the resultant effective weight of R_1. As the water continues on the seepage path, it continues to exert seepage forces in the direction of flow which are proportional to the friction loss per unit of distance. When the cross-sectional area through which flow takes place is restricted, as under the dam, the velocity of the seepage for a given flow is increased. The increase in velocity is accompanied by an increase in friction loss, and the seepage force is correspondingly increased. This increase is represented in figure 98 by the larger vectors for F_2 and F_3 as compared with F_1 and F_4. As the water percolates upward at the downstream toe of the dam, the seepage force tends to lift the soil, reducing the effective weight to R_4. If F_4 exceeded W_s, the resultant would be acting upward and the soil would be floated out. The erosion would progress backwards along the flow line until a "pipe" would be formed to the reservoir, allowing rapid escape of reservoir storage and subsequent failure of the dam.

Experience has shown that this action can be slow and accumulative and the resulting failure can be a sudden upheaval of the foundation at the downstream toe of the dam. Some engineers [7] refer to this type of piping failure as "failure by heave." Others [8] describe this type of piping failure as "blowouts." It does not follow that

piping will always result in a heave or blowout type of failure. If the foundation soil is nonuniform, the fine material may be carried away, leaving the coarse material behind, thus tending to produce a reverse filter which will prevent further piping. Inasmuch as it is difficult to determine whether piping will result in failure or will produce an eventual stabilization in any specific case, it is advisable to design the structure so that piping will not occur.

The magnitude of the seepage forces throughout the foundation and at the downstream toe of the dam where piping must begin depends on the rate of head loss of the percolating water. Relatively impervious foundations or pervious foundations with adequate cutoff trenches are not susceptible to piping because impervious soil offers so much resistance to the flow of water that the reservoir head is largely dissipated in overcoming friction before the downstream toe of the dam is reached; whereas pervious foundations (either homogeneous or stratified) may permit percolation to reach the downstream toe of the dam without a substantial loss of head due to friction. In such instances the designs must be investigated to insure that seepage forces at the downstream toe will not result in blowouts.

Another type of piping failure is due to internal erosion that starts in springs near the downstream toe and proceeds upstream along the base of the dam, the walls of a conduit, a bedding plane in the foundation, an especially pervious stratum, or other weakness which permits a concentration of seepage to reach the area downstream from the dam without high friction losses. This type of failure is termed by some engineers [7] as "failure by subsurface erosion."

The magnitude and distribution of seepage forces in the foundation can be obtained from the flow net, which is a graphical representation of the paths of percolation and lines of equal potential (pressure plus elevation above a datum) in subsurface flow. It consists of flow lines and equipotential lines superimposed upon a cross section of the soil through which flow is taking place. Although the two families of curves may in simple cases be derived mathematically, the graphical solution is more commonly used. The method of drawing and applying the flow net to the solution of problems involving subsurface flow is given in many publications [9, 10, 11].

The analysis of seepage pressures and the safety of the foundation against piping by the flow net method has some serious limitations. It takes considerable experience to draw an accurate flow net, especially where foundations are stratified and where drains or partial cutoffs are installed. For strata and lenses of different permeability, the magnitudes of the permeability coefficient for each layer and in different directions are required. Furthermore, the flow net method of analysis is applicable only to the determination of the safety against blowout piping which, theoretically, is virtually independent of the grain size of the foundation soil and should occur upon first filling of the reservoir. Experience, however, has shown that the grain size and gradation of the foundation material do have an important bearing on piping failures and that piping failures often take place after the dam has been in service for some time. Therefore, it appears that many failures due to piping are of the subsurface erosion type as a result of seepage following minor geological weakness. This type of failure cannot be analyzed by flow nets or other theoretical methods.

For the above reasons and because of the lack of detailed foundation exploration which would make such an analysis meaningful, the construction of flow nets is not required for the design of foundations for small dams. The foundation designs given in the remainder of this chapter are based upon the same theoretical principles which are used in the design of major structures, but the procedures have been simplified so they may be applied to the design of small dams by those who are not specialists in the field of soil mechanics.

126. *Methods of Treatment of Sand and Gravel Foundations.*—(a) *General.*—Various methods of seepage and percolation control can be used, depending on the requirements for preventing uneconomical loss of water and the nature of the foundation as regards stability from seepage forces. Cutoff trenches, sheet piling, mixed-in-place concrete pile curtains, or combinations of these methods have been used to reduce the flow and to control seepage forces. Blankets of impervious material, extending upstream from the toe of the dam and possibly covering all or part of the abutments, are frequently used for the same purpose. Horizontal drainage blankets may be incorporated in the downstream toe of a dam or used to blanket the area immediately downstream from the toe of

the dam through which percolating water may escape under an appreciable head. The purpose of these blankets is to permit free flow and dissipation of pressure without disruption of the foundation structure and loss of fine soil particles. Drainage wells are devices used to relieve pressure in pervious layers which are covered by impervious strata to prevent blowouts downstream from the dam.

The details of these various devices together with an appraisal regarding their effectiveness are contained in the remainder of this section. The application of the various devices to the design of pervious foundations is included in section 127.

(b) *Cutoff Trenches.*—These may be classified into two general types: sloping-side trenches and vertical-side trenches. Sloping-side cutoff trenches are excavated by shovels, draglines, or scrapers and are backfilled with impervious materials which are compacted in the same manner as the impervious zone of the embankment. Vertical-side trenches are also used as cutoffs and may be excavated in open cut by hand or trenching machine, or by stoping where it is necessary to remove and replace breccia or debris in fault zones. Ordinarily, vertical-side trenches are not economical because of the cost of the hand labor involved in placing and compacting the backfill material.

The cutoff trench should be located well upstream from the centerline of the dam, but not beyond a point where the cover of impervious embankment above the trench will fail to provide resistance to percolation at least equal to that offered by the trench itself. The centerline of this trench should be kept parallel to the centerline of the dam across the canyon bottom or valley floor, but it should converge toward the centerline of the dam as it is carried up the abutments in order to maintain the required embankment cover.

Whenever economically possible, seepage through a pervious foundation should be cut off by a trench extending to bedrock or other impervious stratum. This is the most positive means of controlling the amount of seepage and insuring that no difficulty will be encountered by piping through the foundation or by uplift pressures at the downstream toe.

Figure 99 shows the cutoff trench excavation at the left abutment of Shadow Mountain Dam. The designed width and depth of the cutoff trench were 15 and 27 feet, respectively. The completed dam is shown in figure 94, and the maximum section is shown in figure 150. Figure 100 shows a portion of the cutoff trench for Dickinson Dam, constructed by the Bureau of Reclamation, prior to final cleanup. A maximum section of this dam is shown in figure 137. The cutoff trench for Dickinson Dam was about 20 feet deep and 2,000 feet long, and had a bottom width of 40 feet. The piping shown in the photograph is part of the well-point unwatering system. The placing of compacted fill in the cutoff trench is visible in the middle background of the picture.

In order to provide a sufficient thickness of impermeable material and an adequate contact with the rock or other impervious foundation stratum, the bottom width of the cutoff trench should increase with an increase in reservoir head. However, the cutoff trench bottom width may be decreased as the depth of the trench increases, because the seepage force at the foundation contact will decrease due to loss of head in traveling vertically through the foundation as the depth increases. An adequate width of cutoff trench for a small dam may be determined by the formula:

$$w=h-d \tag{2}$$

where:

w=bottom width of cutoff trench,
h=reservoir head above ground surface, and
d=depth of cutoff trench excavation below ground surface.

A minimum bottom width of 20 feet should be provided so that excavating and compacting equipment may operate efficiently in trenches which, if below the water table, must be unwatered by well points or sump pumps.

(c) *Partial Cutoff Trenches.*—The Darcy formula for seepage, equation (1), indicates that the amount of seepage is directly proportional to the cross-sectional area of the foundation. It may be concluded from this that the amount of seepage could be reduced 50 percent by extending the impervious zone in figure 97 into the ground so that the depth of the pervious foundation is reduced from 70 to 35 feet; however, this is not the case. The action of a partial cutoff is similar to that of an obstruction in a pipe—the flow is reduced because of the loss of head due to the obstruction, but the reduction in flow is not directly proportional to the reduction in the area of the pipe. Experiments by Turnbull and by Creager on homogeneous isotropic pervious foundations have dem-

Figure 99. **Cutoff trench excavation at left abutment of Shadow Mountain Dam, a structure on the Colorado River in Colorado. SM-53-CBT.**

onstrated that a cutoff extending 50 percent of the distance to the impervious stratum will reduce the seepage by only 25 percent; an 80 percent cutoff penetration is required to reduce the seepage 50 percent [12].

A partial cutoff trench may be effective in a stratified foundation by intercepting the more pervious layers in the foundation and by substantially increasing the vertical path the seepage must take. Reliance cannot be placed upon a partial cutoff trench in this situation unless extensive subsurface exploration has verified that the more impervious layers are continuous. Pervious foundations also may consist of an impervious foundation stratum of considerable thickness sandwiched between upper and lower pervious layers, and a partial cutoff extending to the impervious layer would cut off only the upper pervious layer. This would be effective if the thicknesses of the impervious and upper pervious layers are sufficient to resist the seepage pressures existing in the lower pervious layers in the vicinity of the downstream toe so that blowouts do not occur.

(d) *Sheet Piling Cutoffs.*—Sheet piling is occasionally used in combination with a partial cutoff trench as a comparatively economical means of increasing the depth of the cutoff, and under certain conditions it may be used in lieu of a cutoff trench. With few exceptions such piling should be of steel, because of its high strength. Sheet piling cutoffs are practically limited to use in foundations of silt, sand, and fine gravel. Where cobbles or boulders are present, or where the material is highly resistant to penetration, driving or jetting not only becomes difficult and costly, but it is highly doubtful that an effective cutoff can be obtained owing to the tendency of the piling to wander and become damaged by breaks in the interlocks or tearing of the steel. A heavy structural section with strong interlocks should be used if the foundation contains gravel.

It is not practicable to drive sheet piling so that

it is watertight. Under the best conditions, including the use of compound to seal the interlocks and good contact of the bottom of the piling with an impervious foundation, it can be expected that the piling will be 80 to 90 percent effective in preventing seepage. With poor workmanship, or if the piles cannot be seated in an impervious stratum, they will not be more than 50 percent effective.

Figure 100. **Portion of cutoff trench excavation for Dickinson Dam, a small earthfill structure on the Heart River in North Dakota. HD–P–182.**

(e) *Cement-Bound Curtain Cutoff.*—The mixed-in-place cement-bound curtain is a relatively new development which gives promise as an economical means of establishing a cutoff (or partial cutoff) in pervious foundations which do not contain large cobbles and boulders. It has been used recently (1957) in the construction of two Bureau of Reclamation dams, Slaterville Diversion Dam near Ogden, Utah, and Putah Diversion Dam near Sacramento, Calif. Although at this time (1958) there has not been sufficient time to evaluate completely the results for these two dams, indications are that satisfactory cutoffs have been obtained by this method. Sample specifications under which this work was performed are given in appendix G.

The process consists of pumping grout through a hollow rotating drill rod on the end of which is a mixing head. The mixing head has vanes which mix the foundation material with the grout as the head is forced down through it; grout is pumped on both the downward and upward travel to assure thorough mixing. The result is the formation of a cylindrical element of cement-bound sand and gravel. Successive overlapping elements are similarly constructed to form a continuous curtain [13].

Patents on the methods and some of the materials used in the construction of the mixed-in-place cement-bound curtain are held by the Intrusion-Prepakt Co., Cleveland, Ohio. According to their literature, the following depths have been attained with presently available equipment:

Diameter of element, inches:	*Depth, feet*
12	58
18	50
24	30

An overlapping of 18-inch-diameter elements will readily yield a 12-inch-thick curtain, which is the minimum thickness recommended for cutoff purposes.

(f) *Grouting.*—Various materials have been used in attempts to develop grouting procedures which will improve the stability and impermeability of pervious foundations by injecting a substance which will act as a binder and fill the voids. Among these materials have been cement, asphalt, clay, and various chemicals. Cement grouting cannot be successfully used in fine granular materials because of the comparatively large particle size of the cement which limits the penetration. Asphalt grouting is partially limited by the particle size. Clay grouting is of doubtful value because the clay is easily carried away by seepage forces.

Chemical grouting has the advantage of having about the same viscosity as water and, therefore, it can be injected into pervious soils. The most common method involves the use of sodium silicate and a reagent such as calcium chloride. These two chemicals, when mixed in the ground, precipitate and form an insoluble solid gel.

Chemical grouting is a costly process; the injection techniques are complex; and the selection of the grout and appropriate techniques requires

considerable field exploration and laboratory and field testing. Furthermore, the results of the injection process are difficult to evaluate. For these reasons, chemical grouting is not considered to be an appropriate treatment for the foundations of small dams within the scope of this text. For additional information on this subject, the reader is referred to the paper, "Progress Report of the Task Committee on Chemical Grouting" [14] and to the excellent bibliography contained therein.

(g) *Upstream Blankets.*—The path of percolation in pervious foundations can be increased by the construction of a blanket of impervious material connecting with the impervious zone of the dam and extending upstream from the toe. Blankets are usually used when cutoffs to bedrock or to an impervious layer are not practicable because of excessive depth; they are also used in conjunction with partial cutoff trenches. The topography just upstream from the dam and the availability of impervious materials are important factors in deciding on the use of blankets. The blanket may be required only in the stream channel which has been eroded down to sand and gravel, but it may be required on portions of the abutments as well.

Figures 101 and 102 show such an abutment blanket which was constructed during the rehabilitation of Ochoco Dam by the Bureau of Reclamation in 1949. The purpose of this blanket was to reduce the seepage through the landslide debris which forms the right abutment. It was successful in that it reduced the seepage at full reservoir level from 28 to 12 second-feet.

The blanket was made contiguous with the impervious zone of the dam and it extends about 400 feet upstream. The abutment was dressed smooth to receive the blanket, which extends from the reservoir floor to an elevation 53 feet above. The blanket was constructed 5 feet thick normal to the approximate 3 to 1 abutment slope, and is protected from erosion by 2 feet of riprap on 12 inches of bedding. Figure 101 shows the earthfill blanket complete and the beginning of riprap placement. Figure 102 shows a general view of the upstream face of the dam and the right abutment blanket completed.

Areas of the foundation which are covered by a natural blanket should be stripped of trees and other vegetation; defective places should be repaired; and the entire surface of the natural blanket should be rolled to seal root holes and other openings. The stripping of a natural blanket upstream from the dam to secure impervious soil for the construction of the dam should be avoided

Figure 101. **Right abutment blanket construction at Ochoco Dam, on a tributary of the Crooked River in Oregon. Ochoco 51.**

Figure 102. Upstream slope of Ochoco Dam. Ochoco 72.

when a positive cutoff is not provided in the design.

Although blankets may be designed by theoretical means [15], a simplified approach may be used for small dams. A suitable thickness for small dams is 10 percent of the depth of the reservoir above the blanket, with a minimum blanket thickness of 3 feet. These thicknesses are for blankets made from materials which are suitable for the construction of the impervious zone of an earthfill dam and similarly compacted.

The length of the blanket will be governed by the desired reduction in the amount of underseepage. From an examination of equation (1) and figure 97 it is apparent that the amount of seepage is inversely proportional to the length of path (for homogeneous isotropic foundations). The blanket should be extended so that the computed seepage loss is reduced to the amount that can be tolerated from a project use standpoint.

An upstream blanket should not be relied upon to reduce the seepage forces in the foundation to the point that piping failures are precluded. Although theoretically an upstream blanket would accomplish this purpose in a homogeneous foundation, the natural stratification that occurs in almost every alluvial foundation makes it possible for high pressures to exist in one or more strata of the foundation at the downstream toe of the dam. Horizontal drainage blankets or pressure relief devices (drains or wells) should always be provided for a dam on a previous foundation when a complete cutoff trench cannot be secured.

(h) *Horizontal Drainage Blankets and Filters.*—The purpose of a horizontal drainage blanket is to permit the discharge of seepage and to minimize the possibility of piping failures, either of the blowout or subsurface erosion type. It accomplishes this purpose by providing weight over that portion of the foundation downstream from the impervious zone of the dam where high upward seepage forces exist. The blanket must be pervious so that drainage will be accomplished, and it must be designed to prevent movement of particles of the foundation or embankment by seepage discharges.

Horizontal drainage blankets should be incorporated in the design of all small dams on relatively homogeneous pervious foundations where positive cutoff trenches are not provided. They may also be used on relatively homogeneous pervious foundations which are overlain by thin impervious layers; the blanket will supply weight

to stabilize the foundation and will also effectively relieve pressures that may break through the impervious layer. In the case of a stratified pervious foundation without a cutoff trench, the horizontal drainage blanket's effectiveness would not be great because the stratifications hinder drainage in a vertical direction.

Typical horizontal drainage blankets are shown in figure 103. For each case illustrated, the drainage blanket consists of a sloping berm which for economy is incorporated within the downstream toe of the embankment. In (A) and (B) of figure 103, the requirement for drainage is provided by the overlying pervious zone; in (C) of that figure the embankment is homogeneous and a drainage filter is necessary. The filter shown also serves to drain the embankment, and thus makes the dam the modified homogeneous type with the resultant advantages described in section 120.

Figure 103(A) illustrates the recommended minimum length and thickness of a drainage blanket for a zoned embankment having a "minimum" impervious zone for a dam constructed on a pervious foundation without a positive cutoff trench. (The determination of this "minimum" impervious zone is given in section 134(e).) Figure 103(B) illustrates the recommended design for a zoned dam with an impervious core larger than minimum. The reverse slope of the impervious section is a device used to minimize the length of the drainage blanket. The reverse slope facilitates construction of the pervious zone if material is obtained from the cutoff trench excavation, and reduces the amount of embankment. The dashed outline indicates the drainage blanket that would be required if the reverse slope were not used.

The required length of the horizontal drainage blanket may be determined theoretically by means of the flow net (sec. 125(c)), provided the ratio of the horizontal to the vertical permeability of the foundation is known, by means of the procedure known as transformed sections. In this method a drawing is made showing a section through the dam and foundation parallel to the direction of flow. The horizontal scale of the drawing is changed by a factor equal to the square root of the ratio of the vertical to the horizontal permeabilities. The flow net is developed on this

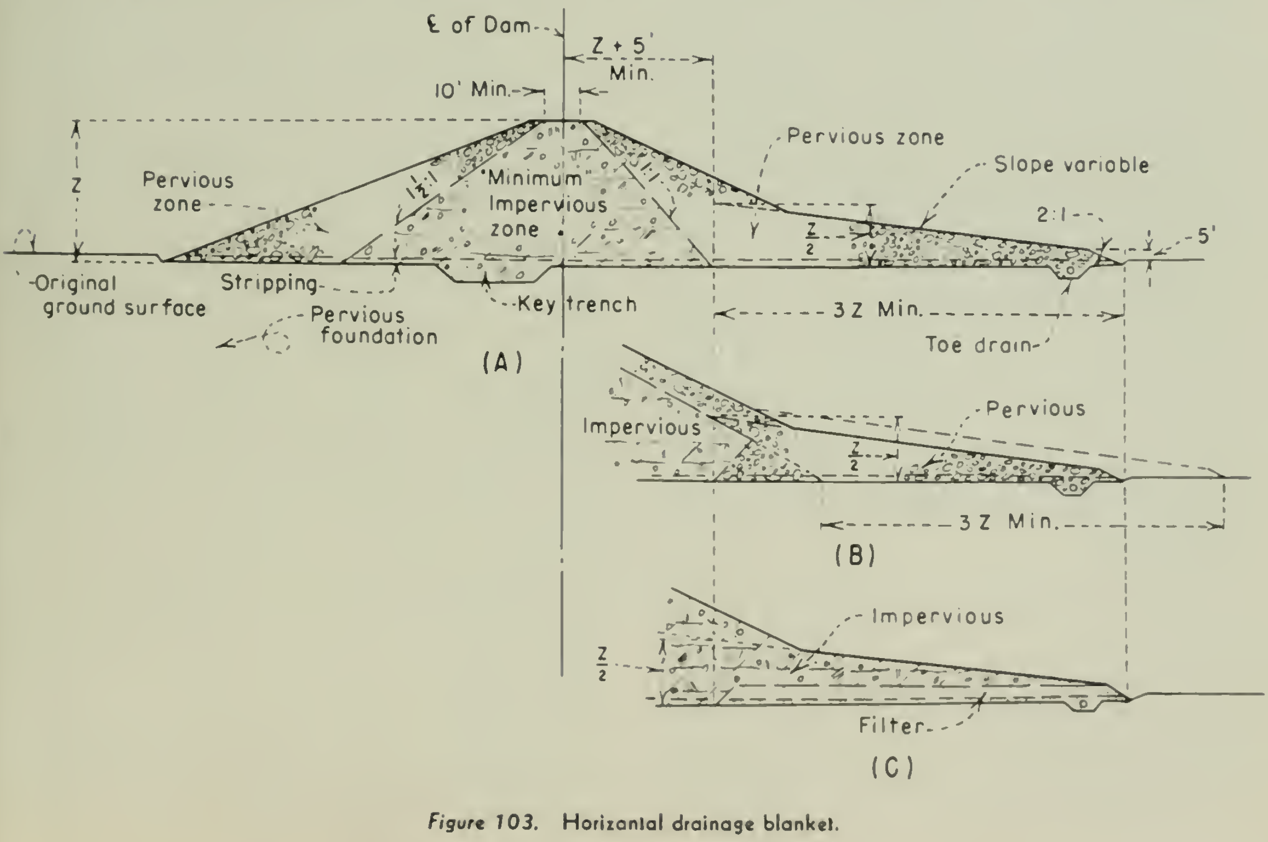


Figure 103. Horizontal drainage blanket.

transformed section as if the foundation were isotropic. After the flow net is completed, it is transferred to the original section drawn without distortion. This in effect changes the "squares" of the flow net to "parallelograms," the horizontal dimensions of which have been determined by multiplying the dimensions of the squares by the square root of the ratio of the horizontal to the vertical permeabilities. This method of dealing with anisotropy in permeable foundations is discussed by Terzaghi and Peck [16]. This method demonstrates that the larger the ratio of horizontal to vertical permeabilities, the farther the seepage emerges downstream from the toe of the impervious zone of the dam and the longer the required drainage blanket.

Because of limitations of the application of the flow net to the design of small dams with relatively meager foundation exploration, the transformed flow net is not required in this text to determine the lengths of horizontal drainage blankets. As a basis for the design for small dams, it is recommended that the length of the blanket be made equal to three times the height of the dam, as shown on figure 103.

The filter shown in figure 103(C) is required to provide drainage. It must be of such gradation that neither foundation nor embankment material particles can penetrate and clog the filter. The filter should have a minimum thickness of 3 feet to provide an unquestionable capacity to conduct seepage flows. Multilayer filters should be avoided, where possible, because of the added expense of such construction.

If the overlying pervious zones in (A) and (B) of figure 103 are sand-gravel similar in gradation to the sand-gravel of the foundation, there is no danger of flushing of particles of the foundation into the embankment, and no special filters are required. If these zones are constructed of rockfill, a filter must be provided so that the finer foundation material is not carried into the voids of the rockfill.

If sufficient quantities of filter material are available at reasonable cost, it usually will be found economical to provide thicker layers than described above rather than to process material to meet the exact requirements for thin filter design, as subsequently described. The thicker the layer, the greater the permissible deviation from the filter requirements given, especially in the requirement of parallelism of gradation curves between filter and base.

The rational approach to the design of filters is generally credited to Terzaghi [17]. Considerable experimentation has been performed by the Corps of Engineers [18] and the Bureau of Reclamation [19]. Several somewhat different sets of criteria are given by these authorities. The following limits are recommended to satisfy filter stability criteria and to provide ample increase in permeability between base and filter. These criteria are satisfactory for use with natural sand and gravel, or with crushed rock, and for "uniform" or "graded" filters:

(1) $\frac{D_{15} \text{ of the filter}}{D_{15} \text{ of base material}}=5$ to 40, provided that the filter does not contain more than 5 percent of material finer than 0.074 mm. (No. 200 sieve)

(2) $\frac{D_{15} \text{ of the filter}}{D_{85} \text{ of base material}}=5$ or less

(3) $\frac{D_{85} \text{ of the filter}}{\text{Maximum opening of pipe drain}}=2$ or more

(4) The grain-size curve of the filter should be roughly parallel to that of the base material.

In the foregoing D_{15} is the size at which 15 percent of the total soil particles are smaller; the percentage is by weight as determined by mechanical analysis. The D_{85} size is that at which 85 percent of the total soil particles are smaller. If more than one filter layer is required, the same criteria are followed; the finer filter is considered as the "base material" for selection of the gradation of the coarser filter.

In addition to the limiting ratios established for adequate filter design, the 3-inch particle size should be the maximum utilized in a filter to minimize segregation and bridging of large particles during placement of filter materials. Also, in designing filters for base materials containing gravel particles, the base material should be analyzed on the basis of the gradation of the fraction smaller than No. 4.

It is important to compact filter material to the same density as that required for construction of sand-gravel zones in embankments, as given in appendix G. Care must be used in placing

filter materials to avoid segregation. The construction of thin filter layers requires proper planning and adequate inspection during placement.

The following is an example (see fig. 104) of a typical design which would be applicable for thin filters, such as those shown around the pipe drains in figure 105.

Given:

Average gradation curve of foundation soil shown on figure 104, with $D_{15}=0.006$ mm. and $D_{85}=0.10$ mm.

Openings in drainpipe, ½ inch.

To find:

Gradation limits of filter materials.

Procedure:

(1) Lower limit of D_{15} of filter $=5\times0.006=0.03$ mm.

(2) Upper limit of D_{15} of filter = *the smaller* of the values: $40\times0.006=0.24$ mm., and $5\times0.10=0.50$ mm.; *use 0.24 mm.*

To meet conditions (1) and (2) and the criterion of parallelism, a sand shown as F_1 in figure 104 was selected. For F_1, $D_{15}=0.14$ mm. and $D_{85}=2.4$ mm. This material is too fine to place adjacent to a pipe with ½-inch openings, since the requirement is for D_{85} of the filter to be at least $2\times½=1$ inch; hence, a second filter layer of gravel or crushed rock is required.

(3) Lower limit of D_{15} of gravel is $5\times0.14=0.70$ mm.

(4) Upper limit of D_{15} of gravel = the *smaller* of the values: $40\times0.14=5.6$ mm., and $5\times2.14=10.7$ mm.; *use 5.6 mm.*

(5) Least D_{85} of gravel $=2\times½=1$ inch $=25.4$ mm.

To meet conditions (3), (4), and (5) and the criterion of parallelism, the gravel shown as F_2 in figure 104 was selected.

The lower and upper limits of the D_{15} for F_1 and for F_2 as well as the lower limit for D_{85} for F_2 are shown on figure 104.

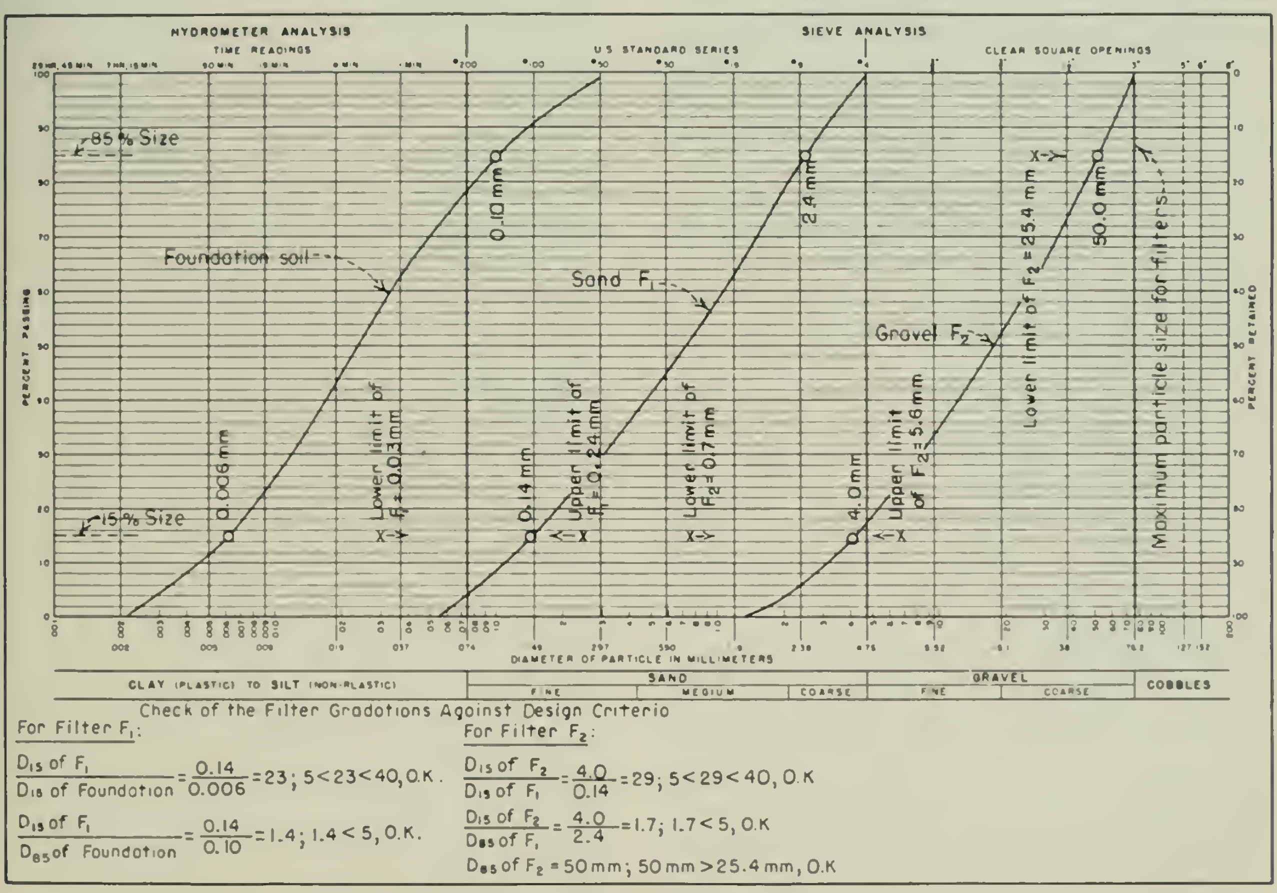
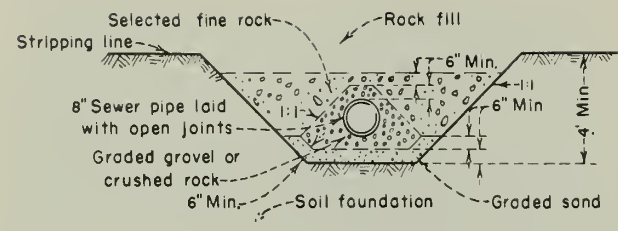


Figure 104. **Typical filter design.**



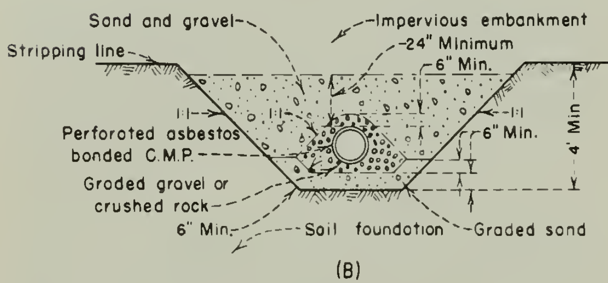


Figure 105. **Typical toe drain installations.**

(i) *Toe Drains and Drainage Trenches.*—Toe drains are commonly installed along the downstream toes of dams in conjunction with horizontal drainage blankets in the position shown in figure 103. Beginning with smaller diameter drains laid along the abutment sections, the drains are progressively increased in size, the lines of maximum diameter being placed across the canyon floor. The purpose of these drains is to collect the seepage discharging from the horizontal drainage blanket and lead it to an outfall pipe which discharges into the spillway stilling basin or into the river channel below the dam. Pipes rather than french drains are used to insure adequate capacity to carry seepage flows. Toe drains are also used on impervious foundations to insure that any seepage that may come through the foundation or the embankment is collected and that the ground water is kept below the surface sufficiently to avoid the creation of unsightly boggy areas below the dam.

The toe drains may be of vitrified clay or concrete tile, or perforated asphalt-dipped corrugated metal pipe. The drainpipes are placed in trenches at a sufficient depth below the ground surface to insure effective interception of the seepage flow. The minimum depth of the trench is normally about 4 feet; the maximum depth is that required to maintain a reasonably uniform gradient though the ground surface undulates. The bottom width of the trench is 2 to 3 feet, depending on the size of the drainpipe. A minimum pipe diameter of 6 inches is recommended for small dams; diameters up to 18 inches may be required for long reaches at flat gradients. The drainpipe should be surrounded by a filter to prevent clogging of the drains by inwash of fine material or piping of foundation material into the drainage system. Two-layer filters are often required; the layer in contact with the pipe must be of sufficient particle size so material will not enter or clog the perforations in metal pipe or the joint openings in clay or concrete tile. The design of filters is discussed in the previous subsection. Figure 105 shows typical installations of toe drains in Bureau of Reclamation dams.

Toe drains may be installed in a pervious foundation overlain by an impervious layer if the impervious layer is not too thick to be penetrated by an open trench. Such a drain is normally called a drainage trench and is effective in relieving uplift pressures in the underlying pervious stratum. A drainage trench usually is not effective if the underlying pervious foundation is stratified, as it will relieve uplift pressures only in the uppermost pervious stratum. More effective drainage of stratified foundations can be accomplished by pressure-relief wells.

(j) *Pressure-Relief Wells.*—In a great number of instances of low dams on pervious foundations overlain by an impervious stratum, the thickness of the top impervious layer is such that there is no danger of piping, either of the blowout or internal-erosion type. Theoretically, piping occurs when the fluid (uplift) pressure at a point at some level in the foundation in the vicinity of the downstream toe reaches the pressure exerted by the combined weight of soil and water above it. For the usual condition of tailwater at the ground surface, the uplift pressure (in feet of water) at the point in question will be equal to the depth, d, of the point below ground plus the reservoir pressure head minus the head lost in seepage through the foundation to the point. The pressure exerted by the weight of soil and water above this point is the saturated unit weight of soil times the depth to the point. If the thickness of the impervious layer is equal to the reservoir head, h, the uplift pressure beneath the layer cannot exceed the weight of the

layer. This is so because the saturated weight of soil equals approximately twice the weight of water, and for $h=d$:

$$2\gamma_w \times d \times 1 = (h+d)\gamma_w \qquad (3)$$

(or pressure exerted by saturated weight = uplift pressure).

Actually, there is always appreciable loss of reservoir head because of seepage friction; hence the value h in the right-hand portion of equation (3) is too large and the uplift pressure will be smaller than the pressure exerted by the overlying weight. Therefore, if the thickness of the top impervious stratum is equal to the reservoir head, it may be considered that an appreciable factor of safety against piping is assured. In this situation no further treatment of the foundation is required. However, if the thickness of the top impervious stratum is less than the reservoir head, some preventive treatment is recommended. If the top impervious stratum is less than h but is too thick for treatment by drainage trenches, or if the pervious foundation is stratified, pressure relief wells are required.

The primary requirements for a pressure relief well system are:

(1) The wells should extend into the pervious foundation underlying the impervious top layer to relieve pressures to such a depth that the combined thickness of the impervious layer and drained material is sufficient to provide stability against underlying unrelieved pressures. Depths of wells equal to the height of the dam will usually be satisfactory.

(2) The wells must be spaced sufficiently close together to intercept the seepage and reduce uplift pressures between wells to acceptable limits.

(3) The wells must offer little resistance to infiltration of seepage and discharge thereof.

(4) The wells must be designed so that they will not become ineffective due to clogging or corrosion.

The U.S. Corps of Engineers has carried out extensive research programs on the design and installation of relief wells. Results of these studies have been published in a number of excellent papers [20, 21, 22]. The reader is advised to consult these references for theoretical design methods.

Well spacing usually must be based on judgment because of the lack of detailed information regarding the foundations of small dams. This is an acceptable procedure provided that plans are made to install additional wells after the dam is constructed at the first signs of excessive pressures. When the pervious strata have high rates of permeability, there will be a greater quantity of water at the downstream toe of the dam than if the permeability rate were lower. Suggested well spacing is 25 feet minimum for the most pervious foundations to 100 feet maximum for less pervious foundations.

Experiments have shown that the well diameter should not be less than 6 inches so that there will be little head loss for infiltrating seepage. It is recommended that a minimum thickness of 6 inches of filter which meets the criteria previously established (sec. 126(h)) be provided between the well screens and the foundation, and that the ratio of the 85 percent size of the filter to the screen opening be greater than 2.0.

A pressure relief well which meets the requirements of offering little resistance to infiltration of seepage and the discharge thereof, and which is constructed of relatively inert materials has been developed by the Corps of Engineers [22], as shown in figure 106. The well consists of a screen section, riser pipe, gravel filter, and sand backfill from the top of the gravel filter to an elevation 10 feet below finished ground surface. The screen and riser are of 8-inch wood pipe. The screen is perforated with slots 3/16 inch wide and 3¼ inches long, and the bottom of the well screen is closed with a wood plug. The top of the well is protected by metal guards. One of the best methods developed for installing such a well is the "reverse rotary" method [22]. This is basically a suction dredging method in which the material in the hole is removed by a suction pipe; the walls of the hole are supported by maintaining a head of water in the hole several feet above the water table.

Another method of installing a pressure relief well requires sinking a casing of appropriate size to the required depth and washing out the soil inside the casing. The assembled well pipe, consisting of the screen and riser, is lowered in the casing and properly alined. The filter is then placed in 6- to 8-inch layers and the casing withdrawn a like amount. This process is repeated until the filter is several feet above the top of the screen section. Above this point, impervious backfill or concrete is placed to prevent leakage

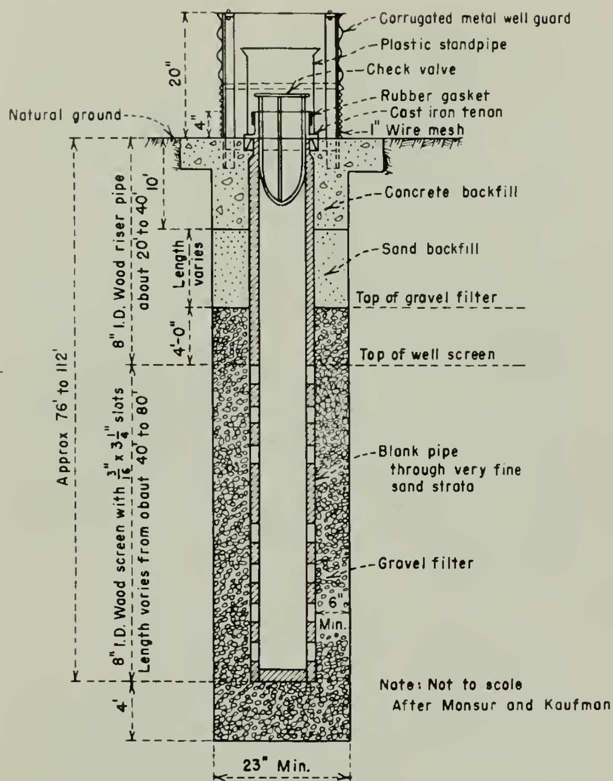


***Figure 106.* Pressure relief well and appurtenances.**

along the outside of the pipe. After installation the wells should be cleaned out and pumped or surged to remove any fine soil immediately adjacent. Pressure relief wells should be inspected periodically and cleaned by surging, if necessary.

Relief wells, like other drainage systems, have some limitations. Too great a volume of seepage may require an excessive number of wells. In such cases upstream impervious blanketing of the areas which provide egress of reservoir water to the pervious layers may be used to reduce the amount of seepage.

Because the design and installation of pressure relief wells demands specialized knowledge, skill, and the highest quality of construction inspection to insure satisfactory results, and because such wells also require postconstruction supervision and maintenance, they are recommended for use with small dams only when other methods of seepage control are not feasible.

127. *Designs for Sand and Gravel Foundations.*—(a) *General.*—One of the criteria for design of earthfill dams given in section 122 requires that the flow of seepage through the foundation and abutments be controlled so that no internal erosion takes place and there is no sloughing in the area where the seepage emerges. The criteria also require that the amount of water lost through seepage be controlled so that it does not interfere with planned project functions. Section 123 discusses the basis used for designing foundations for small dams, which requires a generalization of the nature of the foundation in lieu of detailed explorations and the establishment of design procedures that are less theoretical than those used for major structures. Section 123 also cautions against the use of these design procedures for unusual conditions where procedures based largely on judgment and experience are not appropriate.

The purpose of this section is to show the application of methods of foundation treatment to specific instances. For purposes of discussion, pervious foundations are divided into the following cases:

Case 1. The pervious foundation is exposed.

Case 2. The pervious foundation is overlain by an impervious layer which may vary in thickness from a few feet to a considerable depth.

In both of these cases, the pervious foundation may be relatively homogeneous, or it may be strongly stratified with less pervious layers so that the horizontal permeability will be many times greater than the vertical permeability. Stratification may influence selection of the appropriate foundation treatment method.

The selection of the appropriate foundation treatment for a case 2 foundation also may be influenced by the thickness of the impervious top layer. If the impervious top layer is only a few feet thick, it may be assumed that it will be largely ineffective as a blanket to prevent seepage because thin surface strata usually lack the density required for impermeability and because they commonly have a large number of openings through them. Therefore, a very thin impervious top layer will have little effect on the foundation design. If the thickness of the overlying impervious layer equals or exceeds the reservoir head, it may be considered that there are no problems involved so far as seepage or seepage forces are concerned, as demonstrated by equation (3). In that event the foundation treatment required

will be dictated by the nature of the impervious layer, as regards its settlement and stability characteristics. The designs of impervious foundations of silt and clay are discussed in subsequent sections.

Table 12 is a summary of recommended treatments for various pervious foundation conditions. Included in the table are references to the sections which discuss the designs of the various devices for control of seepage.

TABLE 12.—*Treatment of pervious foundations*

Case	Case No.	Fig. No.	Thickness of overlying impervious layer	Depth of pervious foundation	Stratified or homogeneous	Device for control of seepage	Additional requirements (other than stripping)
Case 1. The pervious foundation is exposed.	1	107(A)	None	Shallow	Either	Cutoff trench (sec. 126(b))	Toe drain (sec. 126(i)). Downstream filter may be required (secs. 126(h) and 127(b)). Grouting may be required (sec. 124).
	1	107(B)	None	Intermediate	Either	Sheet piling or cement-bound curtain cutoff (secs. 126(d) and (e))	Toe drain (sec. 126(i)). Large core required (sec. 134(e)). Key trench (sec. 123). Downstream filter may be required (sec. 126(h) and 127(c)).
	1	108	None	Intermediate or deep	Stratified	Partial cutoff trench (sec. 126(c))	Same as above, except no key trench required.
	1	107(C)	None	Deep	Either	Horizontal drainage blanket (sec. 126(h))	Large core required (sec. 134(e)). Upstream blanket (sec. 126(g)) may be required to minimize seepage loss. Key trench (sec. 123). Toe drain (sec. 126(i)).
Case 2: The pervious foundation is overlain by an impervious layer which may vary in thickness from a few feet to a considerable depth.	2	107(A),(B),(C), 108	Less than 3 feet	Various depths		Treat as corresponding case 1 foundation.	
	2	107(A),(B), 108	More than 3 feet, less than reservoir head	Shallow or intermediate		Treat as corresponding case 1 foundation.	
	2	109 (A) or (B)	More than 3 feet, less than reservoir head	Deep	Homogeneous	Drainage trench (secs. 126(i) and 127(c)) or pressure relief wells (sec. 126(j))	Key trench (sec. 123).
	2	109(B)	More than 3 feet, less than reservoir head	Deep	Stratified	Pressure relief wells (sec. 126(j))	Key trench (sec. 123). Toe drain (sec. 126(i)).
	2		More than reservoir head			No treatment required as pervious foundation.	

(b) *Case 1 Pervious Foundation of Shallow Depth.*—The foundation treatment for this type of situation is shown in figure 107(A). A cutoff trench to the impervious stratum should always be provided in a pervious foundation of shallow depth because it is the most positive means of avoiding excessive seepage losses and avoiding the possibility of piping. A toe drain should be provided for all dams on pervious foundations. If the downstream portion of the embankment is sand and gravel similar in gradation to the foundation, the filter shown would not be required. If the downstream portion of the embankment is rockfill, the filter should extend from the downstream slope of the dam to the impervious zone, as shown. If the embankment is homogeneous, the filter should be provided for the reasons given in section 120(e); however, it need not extend upstream beyond a distance of $z + 5$ feet from the centerline of the dam. Section 134(d) discusses the extent of the filter required for a homogeneous embankment.

(c) *Case 1 Pervious Foundation of Intermediate Depth.*—The treatment for this foundation situation is shown on figure 107(B). This treatment is

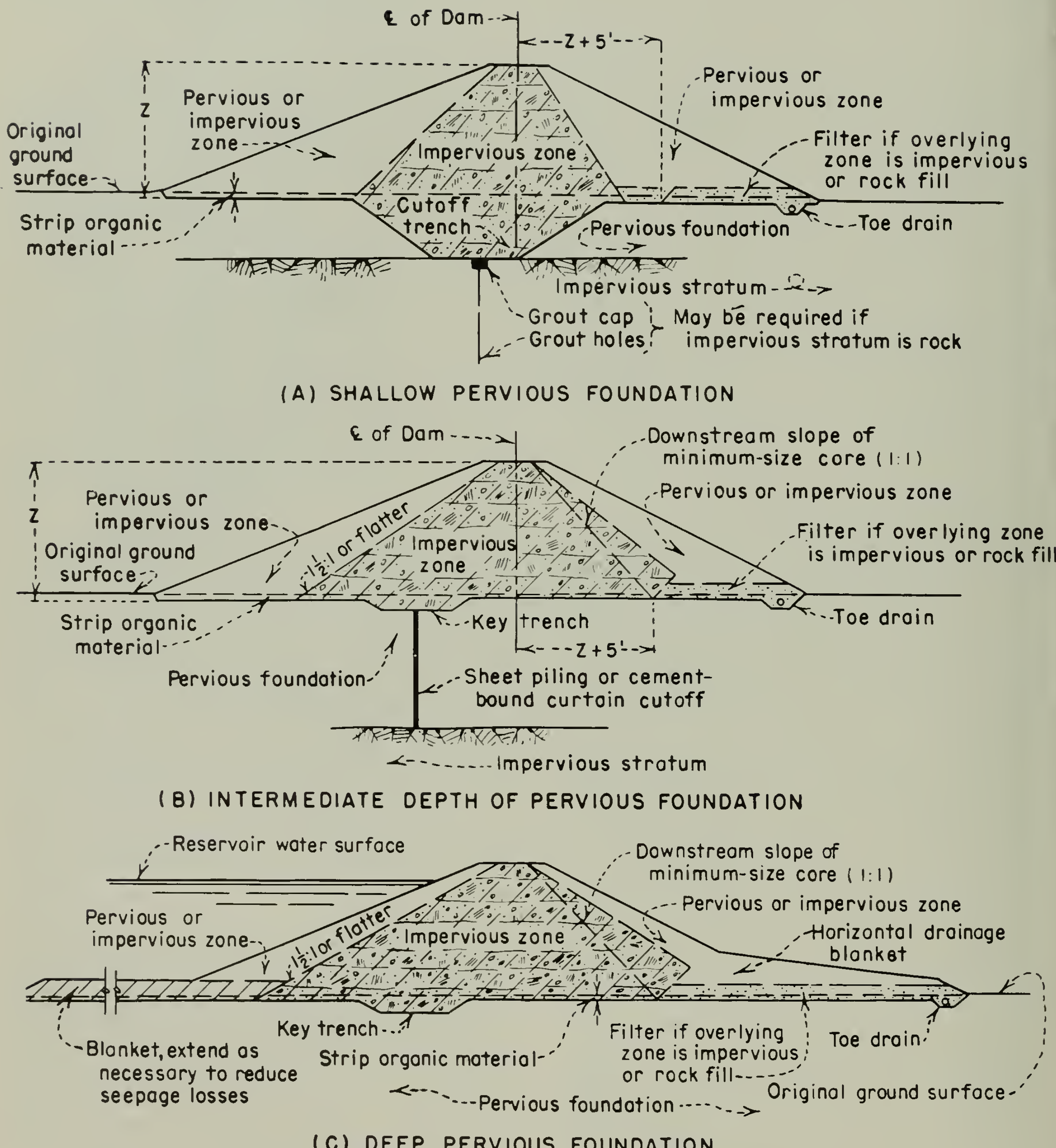


Figure 107. Treatment of case 1 pervious foundations.

appropriate when the depth to an impervious stratum is too great for a cutoff trench but can be reached economically by sheet piling or a cement-bound curtain cutoff. The minimum impervious zone for a zoned embankment constructed on a pervious foundation shown on figure 103(A) is also the minimum that can be used with this design treatment.

If the downstream portion of the embankment is sand and gravel similar in gradation to the foundation, the filter shown would not be required; however, it is required if the embankment is rockfill. If the embankment is homogeneous, the filter should be provided for the reasons given in section 120, and it should extend from the downstream toe of the dam to a distance of $z + 5$ feet from the centerline of the dam. See section 134(d) for a discussion of the filter requirements for a homogeneous embankment. The downstream toe of the dam will to some extent perform the function of a horizontal drainage blanket should the sheet piling or cement-bound curtain cutoff not be completely effective in reducing uplift pressures in the foundation.

If the foundation is stratified, the partial cutoff trench may be used in lieu of the sheet piling or cement-bound curtain cutoff. The partial cutoff trench is effective in controlling seepage pressures only in stratified foundations where there is a marked difference in the permeability of the pervious and relatively impervious layers. However, the relatively impervious layers need not be truly impervious. Also, the partial cutoff trench must extend into the foundation so that the depth to the top of the uppermost pervious layer *which is not cut off* is not less than the reservoir head. In figure 108, d must be equal to or greater than h. (Note that d is the depth to the uppermost pervious layer which is not cut off and is not the depth of the partial cutoff trench.) If this requirement is met, the foundation will be stable against seepage pressures which may exist in the uppermost pervious layer which is not cut off (refer to equation (3)). In other respects the foundation treatment shown on figure 108 is the same as that shown on figure 107(B).

(d) *Case 1 Pervious Foundation of Great Depth.*—The appropriate treatment for a pervious foundation which is too deep to permit a piling or curtain cutoff is shown in figure 107(C). The upstream blanket is provided if required to reduce the amount of seepage loss to tolerable limits. If the upstream zone is pervious, the blanket should extend under the upstream toe of the dam so as to be contiguous with the impervious core. The minimum impervious zone for a zoned embankment constructed on a pervious foundation without a positive cutoff or a larger zone must be used in this design treatment.

The design of the horizontal drainage blanket and filter is shown on figure 103. Care must be taken in making use of the horizontal drainage blanket to insure that an adequate thickness is maintained over the pervious foundation across the valley floor. The length and thickness of the blanket based on the maximum section of the dam may not be adequate for a section away from the stream channel where the height of dam is less. Several sections should be taken along the centerline of the embankment to determine that an adequate blanket is provided for the most critical section. To avoid warping of the

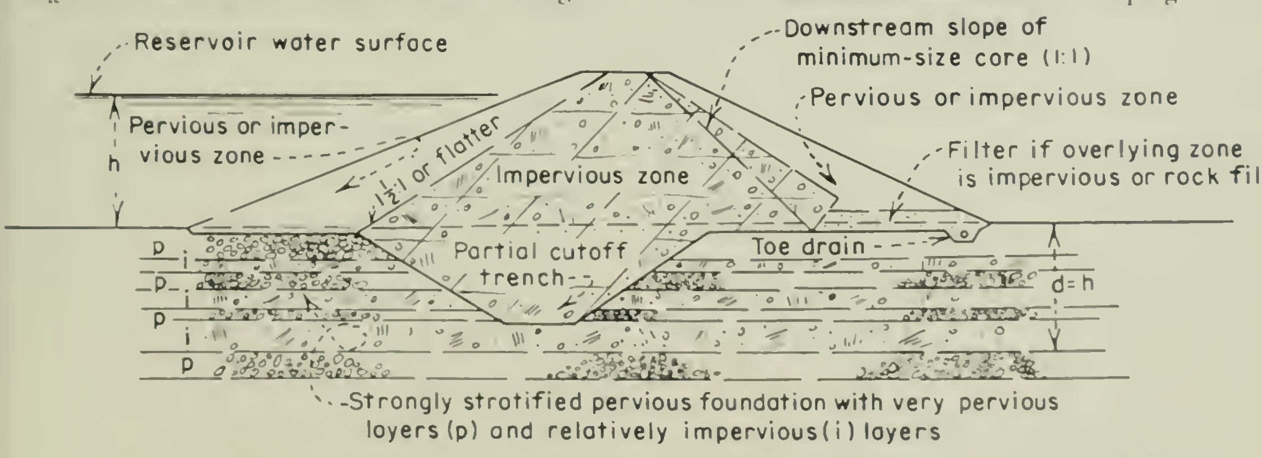


Figure 108. Alternative treatment of case 1 pervious foundations of intermediate or greater depths, if strongly stratified.

downstream slope, the blanket slope required for the most critical section and the corresponding elevation for the intersection of the blanket slope with the downstream slope of the dam can be maintained across the valley floor.

If the foundation is stratified, the partial cutoff trench shown in figure 108 can be used, as discussed in the preceding subsection, in lieu of the horizontal drainage blanket.

(e) *Case 2 Pervious Foundation.*—This subsection is concerned with the treatment of pervious foundations which are overlain by an impervious layer more than a few feet thick but of a thickness less than the reservoir head. If the layer is less than a few feet (say 3 feet) in thickness, its effect may be ignored, and the foundation can be designed as if it were a case 1 foundation. If the layer thickness is greater than the reservoir head, the foundation treatment required, if any, is virtually independent of the underlying pervious foundation for the reason expressed previously (refer to equation (3)).

For thicknesses between the limits stated above, the overlying layer may perform the function of an upstream blanket so that seepage losses will not be excessive. Provisions must be made, however, to safeguard against or relieve the seepage pressures in the underlying pervious foundation; no natural blanket may be assumed to be perfect or so extensive as to prevent seepage or to cause a substantial loss of head for reservoir water entering the underlying pervious stratum.

If the case 2 pervious foundation is of shallow or intermediate depth, the treatment recommended for corresponding case 1 pervious foundations, as shown on figures 107(A), 107(B), and 108 are appropriate. The treatment for a case 2 pervious foundation of great depth depends on the thickness of the overlying impervious layer and whether the pervious foundation is stratified. Figure 109(A) shows the recommended treatment if the top impervious layer is shallow enough to be penetrated by a drainage trench and if the underlying pervious foundation is relatively homogeneous. The drainage trench would not be effective if the pervious foundation were stratified, as it would relieve pressures only in the uppermost pervious layer.

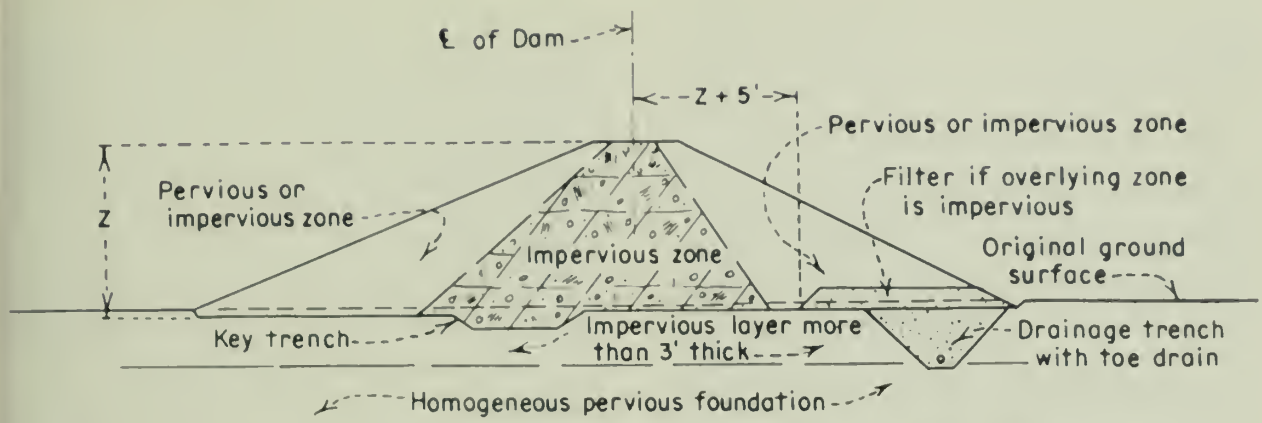
If the impervious top layer is too thick to be penetrated by a drainage trench, or if the underlying pervious foundation is stratified, the recommended foundation treatment is the installation of pressure relief wells, as shown in figure 109(B).

The filters shown in figure 109 are to be provided only to modify homogeneous embankments or embankments which otherwise would not have drainage extending from the downstream toe of the dam to within a distance of $z+5$ feet of the centerline of the dam.

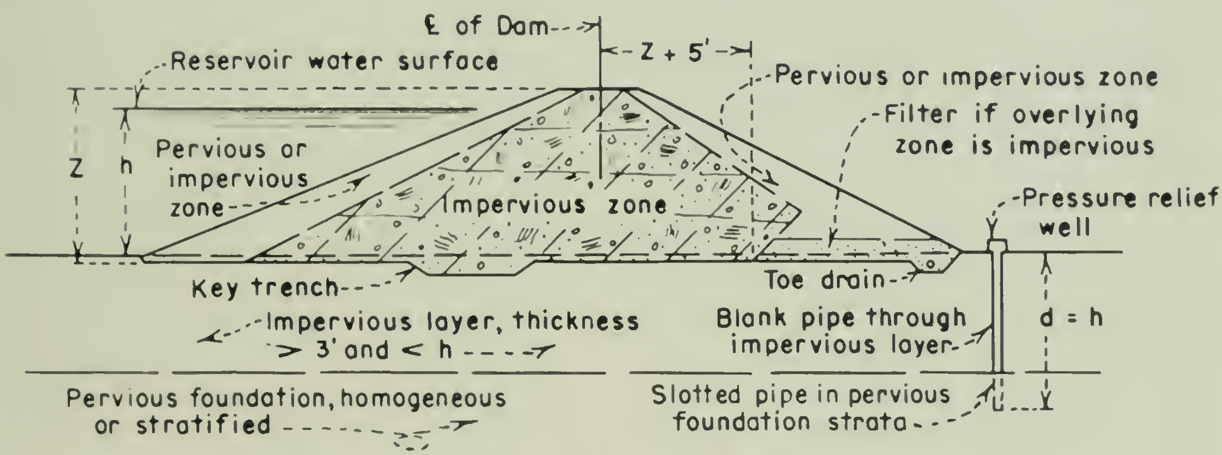
128. *Methods of Treatment of Silt and Clay Foundations.*—(a) *General.*—Foundations of fine-grained soils are sufficiently impermeable to preclude the necessity of providing design features for underseepage and piping. The main problem with these foundations is stability. In addition to the obvious danger of bearing failure of foundations of saturated silts and clays, the designs must take into account the effect of saturation of the foundations of the dam and appurtenant works by the reservoir.

Methods of foundation treatment are based on the soil type, the location of the water table, and the state of compactness of the soil. For saturated foundations of fine-grained soils (including sands containing sufficient fines to make the material impervious), the standard penetration test described in section 103 will provide an approximate measure of their state of compactness or relative consistency. This test cannot be relied on, however, in fine-grained soils above the water table, especially very dry soils whose resistance to penetration is high although their density is low. In these soils the state of compactness can be determined by density inplace tests described in section 114.

(b) *Saturated Foundations.*—When the foundation of an earthfill dam consists of saturated fine-grained soils or saturated impervious sands, their ability to resist the shearing stresses imposed by the weight of the embankment may be determined by their soil group classification and their relative consistency. Soils that have never been subjected to geologic loads greater than the existing overburden are "normally" consolidated. These soils are relatively weak as compared with strata consolidated by hundreds or thousands of feet of ice or soil which has since been removed. Old lake deposits which have experienced cycles of drying and submergence often exhibit the characteristics of preconsolidated soil due to the capillary forces associated with the shrinkage phenomenon. Soils which have been preconsolidated are recog-



(A) OVERLYING IMPERVIOUS LAYER PENETRATED BY DRAINAGE DITCH



(B) PRESSURE RELIEF WELL

Figure 109. Treatment of case 2 foundations with overlying impervious layer of thickness more than 3 feet and less than reservoir head.

nized by their large resistance to penetration, which is usually more than 20 blows per foot; they provide satisfactory foundations for small dams. On the other hand, the presence of soft, unconsolidated silts and clays represented by a penetration resistance of less than four blows per foot indicates the need for special sampling and testing techniques and requires the advice of specialists. By identifying the soil and determining its resistance to penetration, the standard penetration test can be used to delimit those saturated foundations which can be designed by the approximate methods used in this text and to provide approximate design values.

For cohesionless soils the relative density D_d, which equals $\frac{e_{max}-e}{e_{max}-e_{min}}$ (see sec. 115(f)), is known to be related to the strength of the material. For saturated cohesive soils a similar property, the relative consistency C_r, is also related to strength. C_r is equal to $\frac{LL-w}{LL-PL}=\frac{e_{LL}-e_w}{e_{LL}-e_{PL}}$. At water contents equal to their liquid limits ($C_r=O$), the cohesive strength, C'_{LL}, of all remolded saturated

soils is about 0.2 pound per square inch and the shearing strength can be represented by the Coulomb equation:

$$s_{LL}=0.2 \text{ p.s.i.}+\bar{\sigma} \tan \phi_s \quad (4)$$

The value of tan ϕ_s can be obtained by "slow" shear tests on saturated soil starting from the liquid limit condition. Drainage is permitted in these tests and the pore-water pressure is zero. Tan ϕ_s is about 0.5 even for fat clays.

At water contents equal to their plastic limits ($C_r=1.0$), the cohesive strengths of saturated soils vary considerably depending on their types, and the shearing strength can be represented by the equation:

$$s_{PL}=C_{PL}+\sigma \tan \phi \quad (5)$$

The value of tan ϕ on an effective stress basis can be obtained from triaxial shear tests on samples compacted at Proctor maximum dry density and optimum water content. This value is usually somewhat smaller than tan ϕ_s. The value of cohesion at the plastic limit, C_{PL}, can be obtained from similar tests made on soil compacted at optimum water content and then saturated. As explained in section 89(d), for these samples the vertical intercept of the tangent to the failure circle making an angle ϕ with the abscissa on the Mohr diagram (fig. 39) is designated C_{sat}. The water content corresponding to C_{sat} is usually close to the plastic limit for clayey soils; that is, C_r is near unity. By assuming a linear variation of cohesion with water content between the liquid and plastic limits,

$$C_{PL}=\frac{C_{sat}-0.2}{C_r}+0.2 \quad (6)$$

where C_r corresponds to C_{sat}.

Using this assumption, Coulomb's equation for shearing strength (equation (10)) for a saturated soil at any C_r may be written as follows:

$$s=C_{LL}(1-C_r)+C_rC_{PL}+\bar{\sigma} \tan \phi \quad (7)$$

The last term in the foregoing equation represents the frictional portion of the shearing resistance at any point of the potential surface of sliding in the foundation. For the condition of no drainage of the impervious foundation during construction of the embankment, $\bar{\sigma}$ remains constant. The cohesion portion of the equation is a function of C_r. Since C_r cannot increase without drainage, the shearing strength of the foundation remains constant while the shearing stresses imposed by the embankment increase, thus decreasing the factor of safety against sliding. The methods of treatment applicable to these conditions are (1) to remove the soils of low shearing strength, (2) to provide drainage of the foundation to permit increase of strength during construction, and (3) to reduce the magnitude of the average shearing stress along the potential surface of sliding by flattening the slopes of the embankment.

Removal of soft foundation soils is sometimes practicable. Relatively thin layers of soft soils overlying firm material may be removed when the cost of excavation and refill is less than the combined cost of special investigations and the flatter embankment slopes required. In the preparation of relatively firm foundations, pockets of material substantially more compressible or lower in strength than the average are usually removed. See appendix E for a discussion of foundation stripping.

There are several instances where vertical drains have been used to facilitate the consolidation so that the strength of a foundation will increase as it is loaded by an embankment. This treatment is applicable primarily to nonhydraulic structures such as highway embankments. Special studies and precautions are required when these drains are used under an earthfill dam, and this device is not recommended for small dams within the scope of this text.

The most practicable solution for foundations of saturated fine-grained soils is to flatten the embankment slopes. This requires the critical surface of sliding to lengthen, thereby decreasing the average shearing stress along its path and increasing the factor of safety against sliding. The selection of design slopes is discussed in section 129.

(c) *Relatively Dry Foundations.*—Unsaturated, impermeable-type soils are generally satisfactory for foundations of small dams because the presence of air in the soil voids permits appreciable volume change, increase of normal effective stress, and mobilization of frictional shearing resistance without drainage of the pore fluid. That is, for a given void ratio, an impervious soil has greater bearing capacity in the unsaturated condition than in the saturated condition.

In addition, unsaturated soils exhibit the phenomenon of "apparent cohesion" which is the result of less than atmospheric capillary pressures in the water films surrounding the soil particles. The addition of water to these soils first reduces and then destroys the apparent cohesion as saturation is reached. Most soils are sufficiently dense so that reduction of apparent cohesion by saturation will cause no serious difficulties in foundations of small dams. There is, however, an important group of soils which are of low density and subject to large settlements when saturated by the reservoir, although these soils have high dry strength in the natural state. If proper measures are not taken to control excessive settlement, failure of the dam may occur (1) by differential settlement which causes rupture of the impervious portion of the embankment and thus allows breaching of the dam by the reservoir, or (2) by foundation settlement resulting in a reduction of freeboard and overtopping of the dam, although the impervious portion of the embankment deforms without rupturing.

These low-density soils are typified by but not restricted to loess, a very loose, wind deposited soil which covers vast areas of several continents, including North America. True loess has never been saturated and is generally composed of uniform, silt-sized particles bonded together with a small amount of clay. When its water content is low, loess exhibits sufficient cohesive strength to support 100-foot-high earthfills without large settlement. A substantial increase in water content, however, greatly reduces the cohesion and may result in collapse of the loose structure of the soil under the loading imposed by dams only 20 feet high.

The experiences of the Bureau of Reclamation with the construction of dams on loess in the Missouri River Basin are, in part, described in a publication of the American Society of Civil Engineers [23]. Although the properties of other loessial soils may differ from those found in the Missouri River Basin, a discussion of the Bureau's experience may serve as a guide in other areas.

The typical undisturbed Missouri River Basin loess is a tan to light brown, unstratified, lightweight soil containing many root holes and voids. It consists mostly of silt-sized particles bonded together by a relatively small proportion of clay. The appearance of the loess and the range of gradation are shown in figure 110; 75 percent of the samples investigated were "silty loess," 20 percent were "clayey loess," and the remainder were "sandy loess." The density of the loess ranged from a low of 65 pounds per cubic foot in unusual cases to as high as 100 pounds per cubic foot in areas which had been wetted and consolidated, or where the loess had been eroded and redeposited.

With natural water contents of about 10 percent the supporting capacity of the loess is high, regardless of its density. There is little reduction in bearing capacity for water contents up to about 15 percent. Further increase in moisture is accompanied by an appreciable reduction in supporting capacity for low-density loess, while the increase in moisture has little effect on high-density loess.

Several typical laboratory compression curves for test specimens of loess have been plotted in figure 111 as load versus dry density to demonstrate the effect of inplace density and of wetting on compression characteristics. The low-density loess which is not prewetted (curve A) compresses 5 percent under a load roughly equivalent to a 100-foot-high earthfill dam; it compresses an additional 10.5 percent without an increase in load when saturated. The difference between the compression characteristics of low-density loess at the natural moisture and prewetted conditions indicates that dangerous settlement would result even for a 20-foot-high dam. Figure 111 also demonstrates (curve C) that very little postconstruction foundation settlement will take place for a dam constructed on a high-density loess with low natural moisture. Hence, the determination of the inplace density and water content of the loess is of paramount importance in planning its use as a foundation for a dam.

The required treatment of dry, low-density foundations will be dictated by the compression characteristics of the soil. These characteristics are best determined by laboratory tests on undisturbed samples at natural water content to determine whether postconstruction settlement on saturation will be significant (curve A of fig. 111) or of minor amount (curve C of the same figure). For small dams the empirical criteria given in section 129(b) can be used in lieu of laboratory tests.

If the foundation is not subject to appreciable postconstruction settlement upon saturation, little foundation preparation is required. The foundation should be stripped to remove organic material,

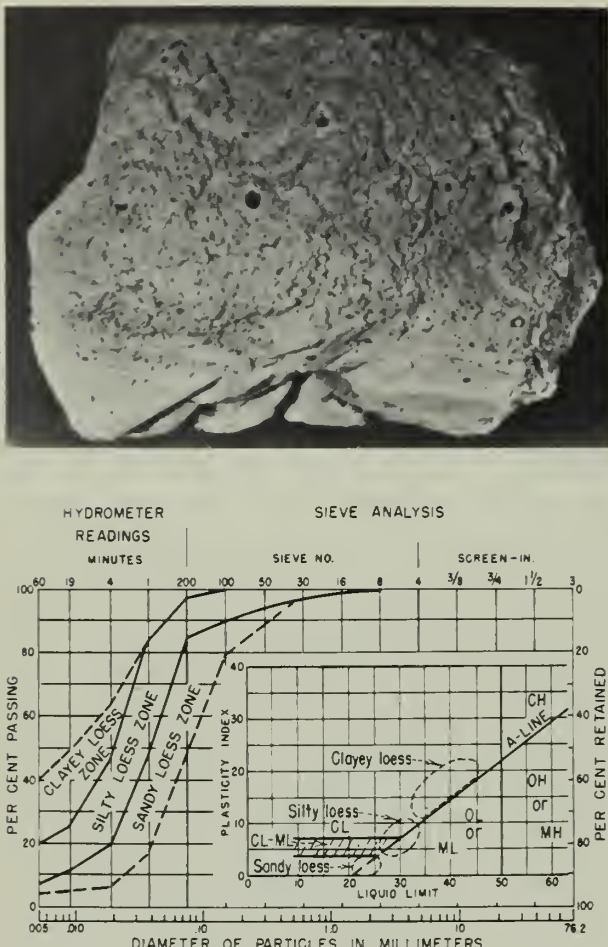


Figure 110. Appearance and identification of Missouri River Basin loess.
(Top) Undisturbed loess (about three-fourths actual size). (Bottom) Range of gradation and Atterberg limits.

After Clevenger [23]

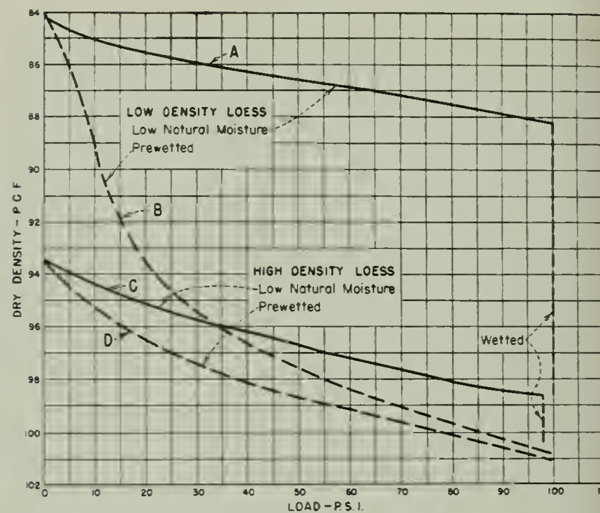


Figure 111. Typical compression curves for Missouri River Basin loess.

a key trench (sec. 123) should be provided, and a toe drain (sec. 126(i)) should be installed to prevent saturation of the foundation at the downstream toe of the dam.

If the foundation is subject to appreciable postconstruction settlement on saturation, measures should be taken to minimize the amount. If the low-density soil exists in a top stratum it may be economical to excavate the material and replace it with compacted embankment. If the layer is too thick for economical replacement or if its removal would destroy a natural blanket over a pervious foundation, measures should be taken to insure that foundation consolidation will be achieved during construction.

Curve B of figure 111 demonstrates that low-density loess, if prewetted, will compress during loading. Hence postconstruction settlement of low-density loess due to saturation by the reservoir can be avoided by prewetting the foundation in order to obtain compression during construction of the embankment. This method cannot be used unless drainage is assured by an underlying pervious layer or unless the deposit is so thick that vertical drainage may take place during compression of the upper portion of the deposit.

Because of its structure and root holes, the vertical permeability of a loess deposit is much higher than its horizontal permeability. The Bureau of Reclamation has successfully obtained consolidation of foundations of low-density loess during construction by prewetting the foundation, with the result that no difficulty has been experienced with postconstruction settlement upon filling of the reservoir. Sample specifications for the performance of this work are included in appendix G.

129. *Designs for Silt and Clay Foundations.*—(a) *Saturated Foundations.*—The designs of small dams on saturated fine-grained soils given in this section are based on the results of numerous stability analyses using various heights of dam and different sets of slopes for the stabilizing fills for each height. Average values of embankment properties were used and the required shearing strength for a safety factor of 1.5 was determined,

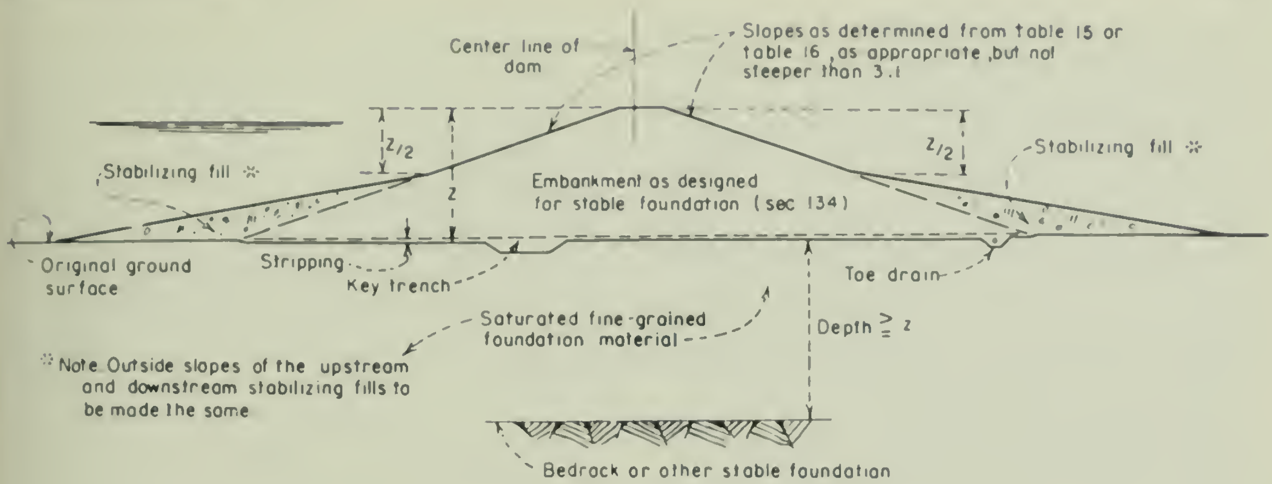


Figure 112. **Design of dam on saturated fine-grained foundation.**

assuming that no drainage occurred in the foundation during construction.

Average strength values of the soil groups were obtained from equation (7). The data from table 6 were used to obtain tan ϕ and to obtain C_{sat} for use in equation (6) to determine C_{PL}. C_{LL} was taken as 0.2 pound per square inch. Figure 112 shows the cross section recommended for use on these foundations. Table 13 gives recommended slopes for stabilizing fills for foundations typical of the groups of the Unified Soil Classification System for different degrees of consistency. Recommendations are not made for slopes for soils averaging less than four blows per foot (standard penetration test) within a foundation depth equal to the height of the dam. These very soft foundations require special sampling and testing which is beyond the scope of this text. Slopes are given for saturated soils of medium consistency (approximately 4 to 10 blows per foot),

TABLE 13.—*Recommended slopes of stabilizing fills for dams on saturated silt and clay foundations*

Consistency	Average number of blows per foot [1] within foundation depth equal to height of dam	Soil group [2]	Slopes for various heights of dam				
			50 feet	40 feet	30 feet	20 feet	10 feet
Soft	Less than 4	Special soil tests and analyses required.					
Medium	4 to 10	SM	4½:1	4:1	3:1	3:1	3:1
		SC	6:1	5:1	4:1	3:1	3:1
		ML	6:1	5:1	4:1	3:1	3:1
		CL	6½:1	5:1	4:1	3:1	3:1
		MH	7:1	5½:1	4½:1	3½:1	3:1
		CH	13:1	10:1	7:1	4:1	3:1
Stiff	11 to 20	SM	4:1	3½:1	3:1	3:1	3:1
		SC	5½:1	4½:1	3½:1	3:1	3:1
		ML	5½:1	4½:1	3½:1	3:1	3:1
		CL	6:1	4½:1	3½:1	3:1	3:1
		MH	6½:1	5:1	4:1	3:1	3:1
		CH	11:1	9:1	6:1	3:1	3:1
Hard	More than 20	SM	3½:1	3:1	3:1	3:1	3:1
		SC	5:1	4:1	3:1	3:1	3:1
		ML	5:1	4:1	3½:1	3:1	3:1
		CL	5:1	4:1	3:1	3:1	3:1
		MH	5½:1	4:1	3:1	3:1	3:1
		CH	10:1	8:1	5½:1	3:1	3:1

[1] Standard penetration tests (sec. 103).

[2] Unified Soil Classification System (sec. 88).

NOTE.—Stabilizing fills are not needed when embankment slopes required by tables 15 and 16 are equal to or flatter than the slope listed above.

stiff consistency (approximately 11 to 20 blows per foot), and hard consistency (greater than 20 blows per foot). When the foundation consists of substantial amounts of more than one group, the slopes selected should be consistent with those recommended in the table.

The stabilizing fills are provided for weight only, and therefore do not require careful selection of materials or special methods of construction. Construction of these fills is described in appendix G under the heading "Miscellaneous fill in dam embankment."

(b) *Relatively Dry Foundations.*—The design of even very small dams on deposits of dry foundations of low density must take into account the possibility of settlement on saturation by the reservoir. Since the penetration test results on these foundations may be grossly misleading, natural water content and density inplace tests should be made in portions of the deposit above the water table for comparison with Proctor compaction values on the same soils. Section 114 describes the procedure for determining inplace density and water content, and section 115 describes the Proctor compaction test. The rapid method of compaction control described in appendix E can also be used to determine the percentage of Proctor maximum dry density existing in the natural soil and the approximate difference between optimum water content and inplace water content.

Analysis of the results of 112 tests made by the Bureau of Reclamation on samples of undisturbed foundation soils indicate that density, water content, and applied load influence the susceptibility of a soil to large settlement on saturation. The following soil groups were represented in the tests: ML, 51 percent; CL, 23 percent; ML–CL, 13 percent; SM, 8 percent; and MH, 5 percent.

For loads within the range applicable for small dams, an empirical relationship between D (inplace dry density divided by Proctor maximum dry density) and $w_o - w$ (optimum water content minus inplace water content) is given in figure 113 which delimits foundation soils requiring treatment from those that do not. There were 70 tests in the former category and 42 in the latter. For foundations of unsaturated soils that fall into the "no treatment required" category in the figure, only the usual foundation stripping and key

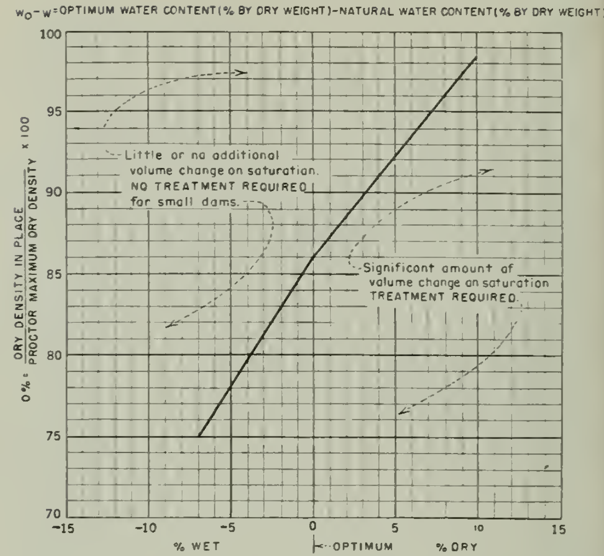


Figure 113. **Foundation design criteria for relatively dry fine-grained soils.**

trench are required. Soils with inplace water content considerably greater than w_o should be checked to determine the degree of saturation. If they are over 95 percent saturated they should be considered as saturated and designed accordingly.

The foundation treatment at Medicine Creek Dam is typical of results achieved by pre-irrigation of a loess foundation. This structure is an earthfill dam located in south-central Nebraska in the center of the Missouri River Basin loess area. As shown on figure 114, dry low-density loess occurred on the right abutment of this dam to maximum depths of 60 to 70 feet, with an average depth of about 40 feet. Undisturbed samples were secured by sinking a test pit at a representative location to a depth of 50 feet. Table 14 summarizes partial results of laboratory tests on these samples. These tests indicated a possibility of dangerous postconstruction settlement upon saturation by the reservoir if the dam were constructed on the natural loess. Therefore, the foundation in this area was thoroughly wetted before fill construction by ponding and sprinkling. Figure 115 shows the dikes and ponds full of water; 33 million gallons of water were used over a 2-month period to raise the water content in the critical area to an average value of 28 percent.

Settlement measuring points throughout the ponded areas revealed that no settlement occurred

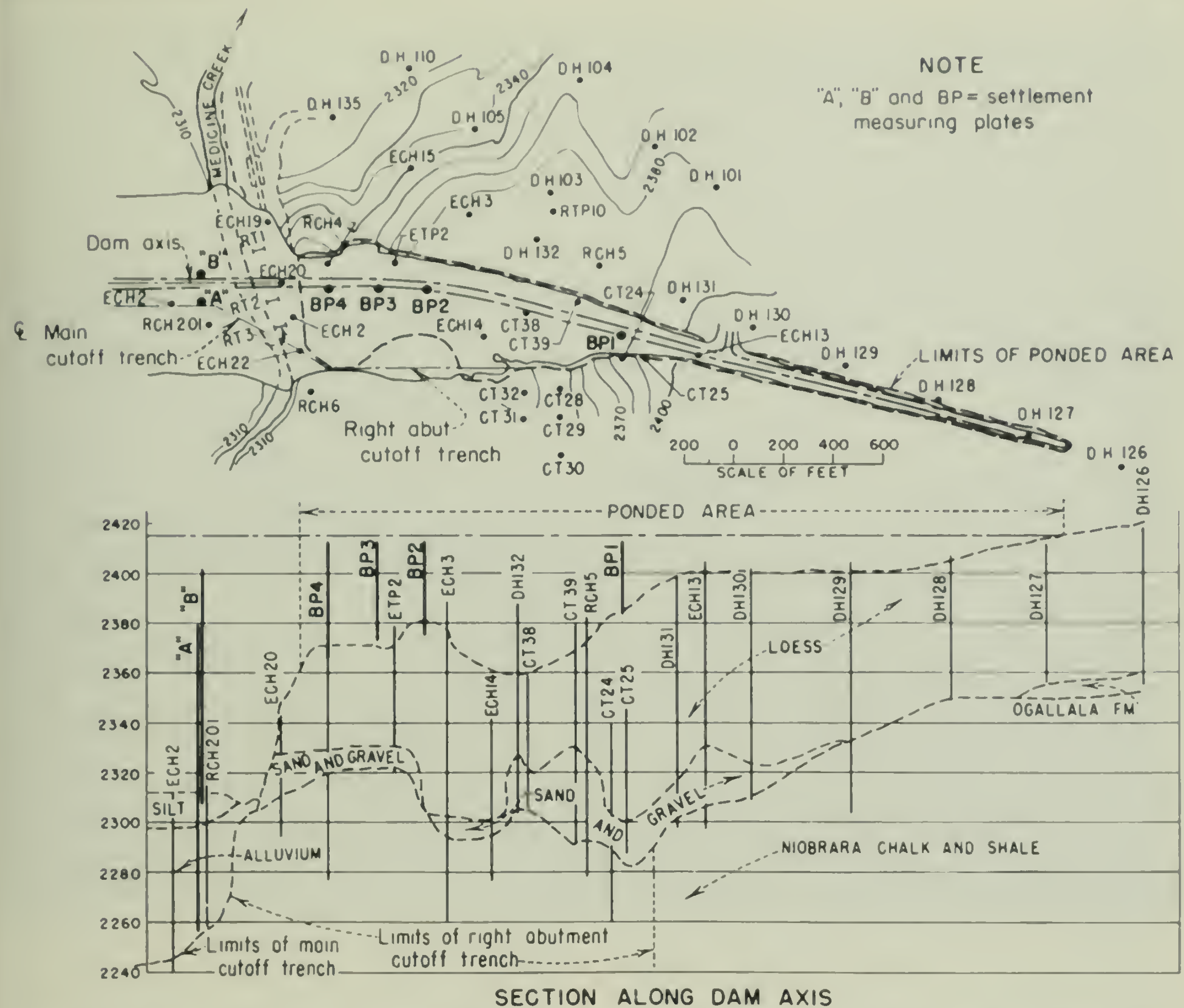


Figure 114. **Geology of right abutment of Medicine Creek Dam, an earthfill structure on Medicine Creek in Nebraska.**

TABLE 14.—*Properties of loess in Medicine Creek Dam foundation*

Approximate sample depth, feet	Natural water content, w, percent	Average w_o-w [1]	Inplace dry density, p.c.f.	Average D (%) [2]	Total load, fill plus overburden, p.s.i.	Compression at total load, percent	
						With natural moisture	After being wetted
5	8.8	7.4	79	75	25	8.4	10.9
17	9.7	6.5	77	74	33	1.3	9.1
19	9.6	6.6	81	77	34.5	1.0	3.9
50	6.6	8.7	92	83	55	1.5	6.5

[1] w_o-w = optimum water content for Proctor maximum dry density minus natural water content.

[2] $D\ (\%) = \dfrac{\text{Inplace dry density}}{\text{Proctor maximum dry density}} \times 100$.

from saturation alone. Base plate apparatus was installed in the dam to permit measurement of foundation settlement in four locations as the fill was constructed (BP1 to BP4 inclusive, fig. 114). The foundation settlements recorded by the base plate installations are shown in figure 116. Upon completion of the embankment in the fall of 1949, the apparatus indicated a foundation settlement of from 0.41 to 0.66 foot. By mid-1952 the measured foundation settlement ranged from a maximum of 2 feet at BP1 to 0.8 foot at BP4. There was virtually no further increase in the amount of settlement by mid-1954 when measurements were discontinued. The reservoir filled to elevation 2366 (normal water surface) in the

Figure 115. **Ponding on foundation of Medicine Creek Dam. 404–1227B.**

spring of 1951 and remained close to that elevation during the period of measurement.

The amount of foundation settlement at Medicine Creek Dam was appreciable, although less than had been anticipated. The foundation consolidation treatment was successful in that a large portion of the settlement took place while the embankment was being constructed, and the subsequent settlement was a slow consolidation over a 2-year period which allowed the dam embankment to undergo the deformation without distress.

D. EMBANKMENTS

130. ***Fundamental Considerations.***—Essentially, the design problem for an earthfill dam embankment is to determine that cross section which, when constructed with the available materials, will fulfill its required function with adequate safety at a minimum cost. The designer of an earthfill dam cannot rely on the application of mathematical analyses or formulas to determine the required cross section to the same degree that one can for a concrete dam. Soils occur with infinite combinations of size gradation, composition, and corresponding variation in behavior under different conditions of saturation and loading; further, the stress-strain relationships in an embankment are very complex.

Considerable progress has been made in investi-

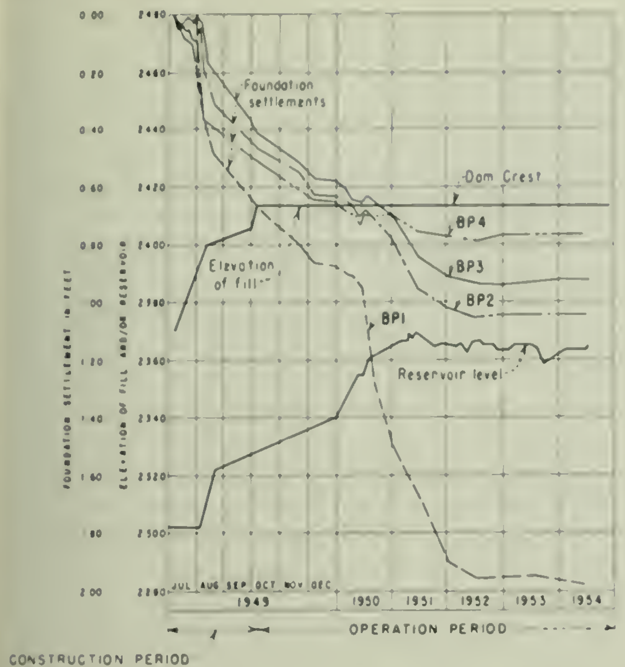


Figure 116. **Record of loess foundation settlement at Medicine Creek Dam.**

gations and studies directed toward the development of methods that will afford a comprehensive analysis of embankment stability. These methods provide useful design tools, especially for major structures where the cost of detailed explorations and laboratory testing of available construction materials can be justified on the basis of economies achieved through precise design. Even so, present practice in determining the required cross section of an earthfill dam consists largely of designing to the slopes and characteristics of existing successful dams, making analytical and experimental studies for unusual conditions, and controlling closely the selection and placement of embankment materials. While some modifications are necessarily made in specific designs to adapt them to particular conditions, radical innovations are avoided and fundamental changes in design concepts are developed and adopted gradually through practical experience and trial.

Although the above practice may be criticized as being overly cautious and extravagant, no better method has yet been conclusively demonstrated. Where consideration is given to the possible loss of life, the certainty of costly property damage in many cases, and the waste of money incidental to the failure of a constructed dam, ample justification is provided for conservative procedures. For small dams, where the cost of explorations and laboratory testing of embankment materials for analytical studies together with the cost of the engineering comprise an inordinate proportion of the total cost of the structure, the practice of designing on the basis of successful structures and past experience becomes even more appropriate.

The design criteria for embankments are given in section 122. They require that the slopes of the embankment be stable under all conditions of construction and reservoir operation; that excessive stresses not be induced in the foundation; that seepage through the embankment be controlled; that the embankment be safe against overtopping; and that the slopes be protected against erosion. This part of the chapter is concerned with the design of the embankment for slope stability and for control of seepage through the dam. Designs of embankment details, such as crest width, freeboard, slope protection, and drainage, are discussed in part E.

The stability of an embankment is determined by its ability to resist shearing stresses, since failure occurs by sliding along a shear surface. Shearing stresses result from externally applied loads, such as reservoir and earthquake, and from internal body forces caused by the weight of the soil and the embankment slopes. The external and internal forces also produce compressive stresses normal to any potential sliding surface. These compressive stresses contribute both to the shearing strength of the soil and to the development of destabilizing pore-water pressures.

Embankments of granular or noncohesive materials are more stable than those made of cohesive soils, because granular materials have a higher frictional resistance and because their greater permeability permits rapid dissipation of pore-water pressures resulting from compressive forces. Accordingly, when other conditions permit, somewhat steeper slopes may be adopted for noncohesive soils. Embankments of homogeneous materials of relatively low permeability have slopes generally flatter than those used for zoned embankments, which have free-draining outer zones supporting inner zones of relatively impervious materials.

In brief, it may be stated that the design of an earthfill dam cross section is controlled by the

physical properties of the materials available for construction, by the character of the foundation, by the methods of construction that are specified, and by the degree of construction control that is anticipated.

131. *Pore-Water Pressure.*—In 1936, Terzaghi [24] demonstrated that in impervious soils subjected to load, a total stress normal to any plane is made up of an effective stress and a fluid pressure. The concepts of plane surfaces and stresses at a point in soils are not identical with those of an ideal homogeneous isotropic material. The "plane" in soils is a rather wavy surface, touching the soil particles only at their contacts with one another; and the "point" of stress is a small region containing enough of the particles to obtain an average stress. With these qualifications the total normal compressive stress, σ, along a plane in an earth structure can be written:

$$\sigma=\bar{\sigma}+u \tag{8}$$

where u is pore-water pressure. From considerations of equilibrium [25], the shearing stress τ along the plane is:

$$\tau=\frac{\sigma_1-\sigma_2}{2}\sin 2\theta \tag{9}$$

where:

σ_1 is the total maximum principal stress,
σ_2 is the total minimum principal stress, and
θ is the angle between the plane considered and the plane on which σ_1 acts.

It is apparent from equation (9) that the shearing stress is the same whether σ_1 and σ_2 are used or their effective components $\bar{\sigma}_1$ and $\bar{\sigma}_2$.

The shearing strength along a plane can be obtained from Coulomb's equation:

$$s=C+(\sigma-u)\tan\phi \tag{10}$$

which shows that the frictional portion of the resistance along a plane is reduced by pore-water pressure. This equation is discussed and the terms are defined in section 89(b).

Pore-water pressures in compacted cohesive soils caused by compressive stresses occur in the sealed triaxial shear test in the laboratory and in the impervious zone of an embankment during construction. For the laboratory conditions the relation between volume change and fluid pressure in a loaded soil mass consisting of solid particles, water, and air, can be derived by using Boyle's law for compressibility of air and Henry's law for solubility of air and water both at constant temperature. For a soil mass buried in an "impervious" fill where drainage is extremely slow because of the long path of percolation and the very small coefficient of permeability of the material, it is both conservative and reasonable, on the basis of field observations, to use the assumption of no drainage to estimate the magnitude of pore-water pressure for design and control purposes [26]. The concept is that when the moist soil mass is loaded without permitting air or water to escape, part of the load causes the soil grains to deform elastically or to undergo nonelastic rearrangement, but without significant change in their solid volume. This part of the load is carried on the soil skeleton as effective stress. The remaining portion of the load is carried by stress in the air and water contained in the voids and is known as pore-water pressure.

Analysis shows that the magnitude of pore-water pressures from compressive forces depends on the compressibility of the compacted soil and on the amount of air it contains. For given conditions of compressibility and loading, the closer the compacted soil is to saturation, the higher the pore-water pressure will be. This leads to the practice of controlling the water content of materials in order to increase the amount of air in the compacted soil. The water content has been reduced below the optimum water content for compaction at Proctor maximum density in the construction of high earthfill dams, but this procedure is neither necessary nor desirable for the construction of embankments less than 50 feet high. For these heights, compaction of cohesive soils at optimum water content and approximately Proctor maximum dry density will insure sufficient air, even in the most compressible soils, to preclude the development of pore-water pressures of appreciable magnitude. For small confining loads, placing material dry of optimum is undesirable because it increases the dangers of (1) low density for the same compactive effort due to the shape of the compaction curve (fig. 90), (2) increased permeability of the embankment, and (3) excessive softening and settlement on saturation by the reservoir, resulting in possible cracking of the fill. On the other hand, the water content should not be appreciably greater than optimum for

Proctor maximum dry density because difficulties have been experienced with unstable fills when very wet soils are used, even in dams of low height.

The foregoing considerations result in the recommended practice of compacting cohesive soils in the cores of small dams close to the optimum water content at Proctor maximum dry density.

132. *Seepage Through Embankments.* The core or water-barrier portion of an earthfill dam provides the resistance to seepage which creates the reservoir. Although, as pointed out in section 89(b), soils vary greatly in permeability, even the tightest clays are porous and cannot prevent water from seeping through them.

The progress of percolation of reservoir water through the core depends on the constancy of reservoir level, the magnitudes of permeability of the core material in the horizontal and vertical directions (anisotropy), the amount of remaining pore-water pressures caused by compressive forces during construction, and the element of time. Figure 117 shows the penetration of water into a core shortly after the first filling of the reservoir, and also the penetration when the steady-state seepage condition has finally been reached. The upper surface of seepage is called the phreatic (zero pressure) surface; in a cross section it is referred to as the phreatic line. Although the soil may be saturated by capillarity above this line, giving rise to a "line of saturation," seepage is limited to the portion below the phreatic line.

The position of the phreatic line depends only on the geometry of the section. For soils of vastly different permeabilities but of the same ratio of horizontal to vertical permeability, the phreatic lines eventually will reach identical positions. It will take much longer for the steady-state condition to be reached in clay than in sand for the same cross section, and the amount of water emerging at the downstream slope will, of course, be much greater in the more pervious material. The pore-water pressures below the phreatic line reduce the shearing strength of the soil mass in accordance with Coulomb's law, equation (10). The steady-state condition, which involves the maximum saturation of the embankment, is the most critical postconstruction condition for the stability of the downstream slope.

The most critical operating condition so far as the stability of the upstream slope is concerned is a rapid drawdown following a long period of high reservoir level. Figure 118 shows the effect of rapid drawdown on the pore-water pressures measured in Alcova Dam, Wyo. It will be noted that the reservoir water surface was lowered 120 feet in 40 days, which is an extremely rapid drawdown for a dam of this height. Figure 118(A) shows the phreatic line and equal-pressure lines under full reservoir conditions; the position of the phreatic line indicates that virtually steady-state conditions were present prior to drawdown. Figure 118(B) shows the pressures under drawdown conditions.

Figure 118 demonstrates that appreciable pore-water pressures remain in an embankment after drawdown. If a specific dam is subject to rapid drawdown after long-term storage at high reservoir levels, special provisions should be made in the designs. The upstream slope of an embankment with an appreciable upstream previous zone usually is not critical for the rapid-drawdown condition. Rapid drawdown may require a flatter slope of a homogeneous embankment than would otherwise be needed for stability.

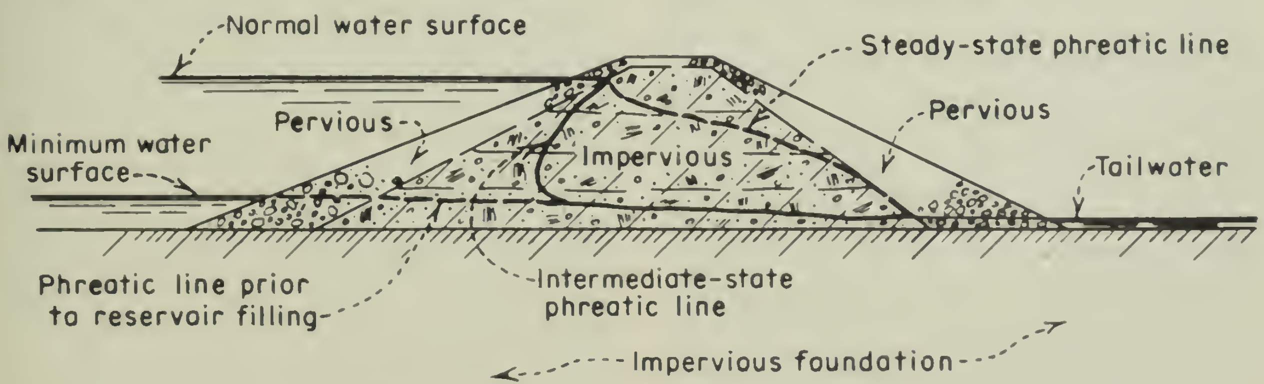


Figure 117. Position of phreatic line in a zoned embankment.

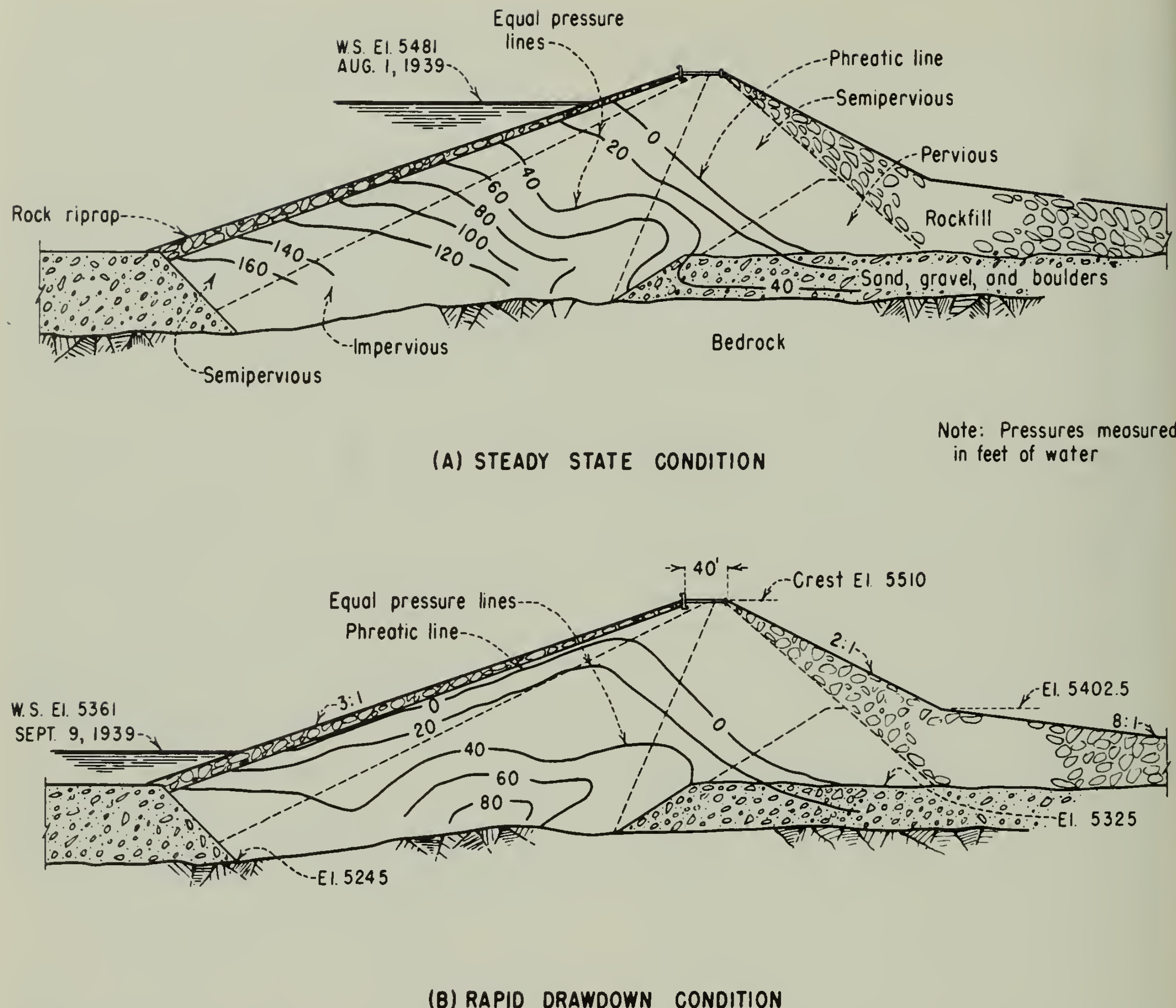


Figure 118. **Effect of rapid drawdown on pore pressures at Alcova Dam, an earthfill structure on the North Platte River in Wyoming.**

The use of the flow net in determining the magnitude and distribution of seepage pressures in pervious foundations has been previously described (sec. 125(c)). The flow net is also used as a means of visualizing the flow pattern of percolating water through embankments in order to estimate the magnitude and distribution of pressures due to percolating water, both in the steady state and in the drawdown condition. Analytical methods of stability analyses which are used in the design of major structures require that such pore-water pressures be determined quantitatively. Such a determination is not required for the design procedure given in this text.

133. ***Stability Analyses.***—Various methods have been proposed for computing the stability of earthfill dams. In general, these methods are based on the shearing strength of the soil and certain assumptions with respect to the character of an embankment failure. The Swedish or "slip-circle" method, which supposes the surface of rupture to be a cylindrical surface, is a comparatively simple method of analyzing embankment stability. Although other and more strictly mathematical solutions have been developed, the slip-circle method of stability analysis is the one most generally accepted. In this method the factor of safety against sliding is defined as the

ratio of the average shearing strength, as determined from equation (10), to the average shearing stress determined by statics on a potential sliding surface.

The force exerted by any segment within the slip-circle is equal to the weight of the segment and acts vertically downward through its center of gravity. The components of this weight acting on a portion of the circle are the force normal to the arc and the force tangent to the arc, as determined by completing the force triangle with lines in the radial and tangential directions. Pore-water pressures acting on the arc result in an uplift force which reduces the normal component of the weight of the segment. Graphical means have been developed by May [27] to facilitate the solution.

The safety factor against sliding for an assumed circle is computed by the equation:

$$\text{Safety factor} = \frac{CL + \tan\phi(N-U)}{T} \tag{11}$$

where:

- N = summation of normal forces along the arc,
- U = summation of uplift forces due to pore-water pressure along the arc,
- T = algebraic summation of tangential forces along the arc,
- L = length of arc of slip circle, and
- C and $\tan\phi$ are as defined for equation (10).

Various centers and radii are used and computations repeated until the arc which gives the minimum safety factor is established.

In order to compute the safety factor by means of equation (11), it is necessary to establish the cohesion and angle of internal friction of the soil, and the magnitude of pore-water pressures for construction, steady-state, and drawdown conditions. Furthermore, the strength properties of the foundation must be determined where the overburden above bedrock is silt or clay, as experience has shown that the critical circle will extend into the foundation in such cases. It is therefore apparent that this method of analysis is more suited to the design of major structures, where the cost of foundation exploration and laboratory tests of foundation and embankment materials to determine their average strength properties is justified because of economies which may be achieved by the use of more precise slopes. The recommended designs for small earthfill dams given in this text are based on the Swedish slip-circle method, using average values of soil properties and experience. These designs will result in adequate factors of safety provided proper construction control is obtained.

134. *Embankment Design.*—(a) *Utilization of Materials from Structural Excavation.*—In the discussion of design criteria (sec. 122), it was pointed out that for minimum cost the dam must be designed to make maximum utilization of the most economical materials available, including material which must be excavated for its foundation and for the appurtenant structures. When the yardage from these sources constitutes an appreciable portion of the total embankment quantity, it may strongly influence the design of the dam. Although these materials frequently are less desirable than soil from available borrow areas, economy requires that they be employed to the maximum practicable extent. Available borrow areas and structural excavations must be considered together in arriving at a suitable design.

The portion of the cutoff excavation above ground-water table may provide limited amounts of material for the impervious core of the dam. Appreciable quantities of sand and gravel are usually obtained in the dewatered portion of the trench from the strata that are being intercepted. When sand and gravel occur in thick, clean beds, they can be salvaged for use in the outer sections of the dam; however, pockets or lenses of silt and clay and highly organic material are often found. In normal excavation operations, the latter materials contaminate the clean soils, which result in wet mixtures of variable permeability and poor workability. Such mixtures will usually be wasted.

Excavations for the spillway usually provide both overburden soils and formation bedrock. In planning the use of these materials, the designer must recognize that moisture control, processing, and special size requirements will add to the cost. For these reasons, material from spillway excavations ordinarily is used only to a limited extent in the main structural zones of dam embankments.

The feasibility of using materials from structural excavation is influenced by the sequence of construction operations. Topography at the dam site, hydrology of the watershed, climate, and the

magnitude of the work all affect the construction sequence and time schedule. An adequate placing area must be available in order to use material from the spillway or cutoff trench in the embankments without having to stockpile and later rehandle large quantities of earth and rock. The placing area is usually restricted early in the job; hence, the designer is faced with the choice of specifying that spillway excavation be delayed until space is available for it, of requiring extensive stockpiling, or of permitting large quantities of material to be wasted. The amount of embankment space that can be provided during the early stages of construction depends in part on the diversion requirements for a particular dam site and in part on the diversion plan that the contractor will select. Usually the contractor is allowed considerable flexibility in the method of diversion; this adds to the designer's uncertainty in planning use of materials from structural excavations. Several plans may have to be worked out to determine the economical choice.

The zoning of the embankment should be based on the most economical utilization of materials which can be devised. An important use of materials from structural excavation has been in portions of the embankment where the permeability was not critical and where weight and bulk were the major requirements. Suitable locations for these miscellaneous fills are the flattened toes of dams on weak foundations where high shearing strength of the fill is not required. These stabilizing fills are discussed in section 129.

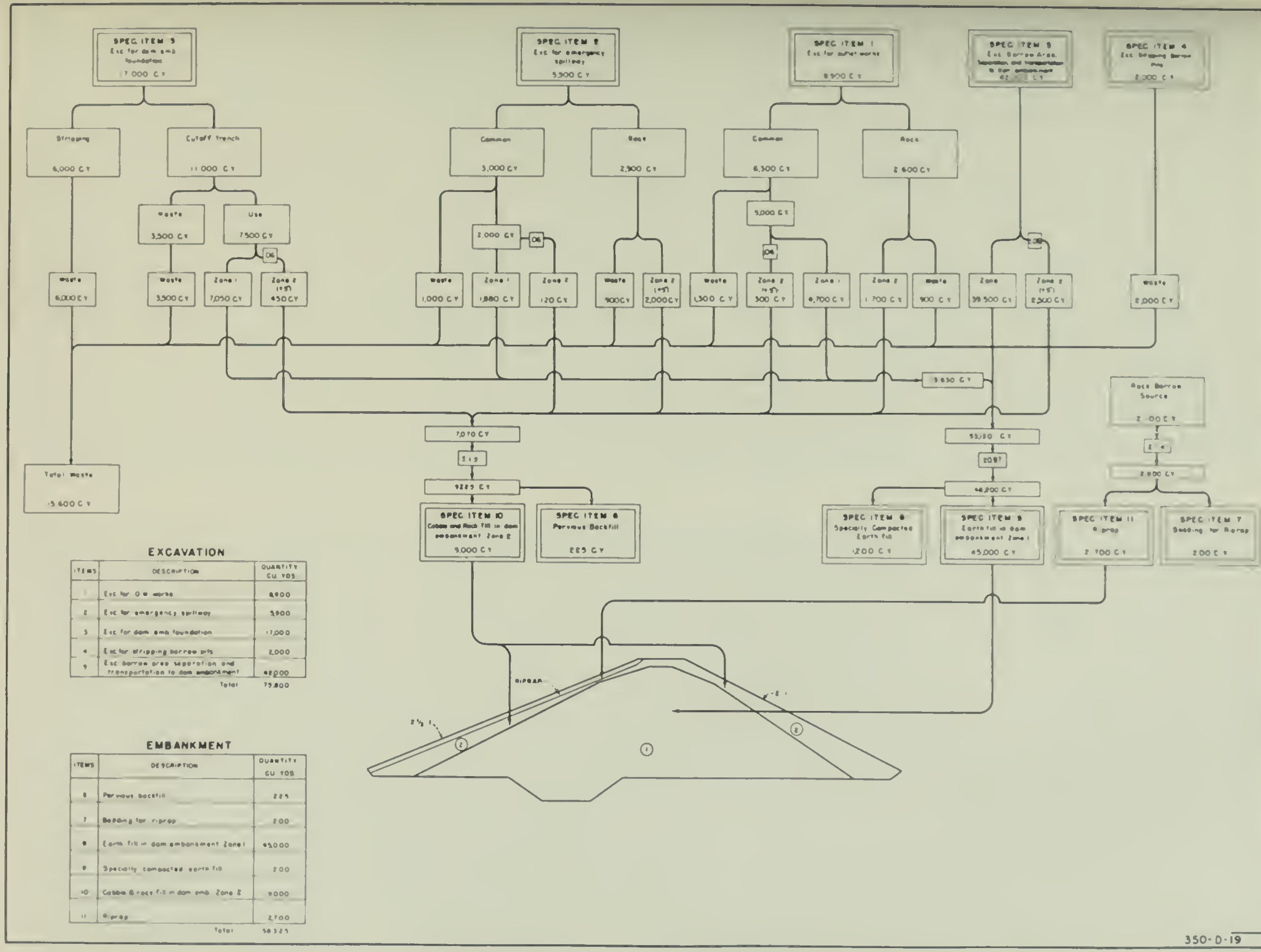
In formulating a design, the designer must estimate the proportion of the structural excavation that will be suitable in the various zones of the embankment, and the shrinkage and swell which will be involved. The use of a materials distribution chart, such as shown in figure 119, has been found helpful. This chart is for the Bureau of Reclamation's Wasco Dam, the maximum section of which is shown in figure 154. In addition to showing the sources of all fill materials, the chart contains the assumptions used for shrinkage, swell, and yield on which specifications quantities are based.

(b) *Embankment Slopes, General.*—The design slopes of an embankment may vary widely, depending on the character of the materials available for construction, foundation conditions, and the height of the structure. The embankment slopes as determined in this section are the slopes required for stability of the embankment on a stable foundation. Pervious foundations may require the addition of upstream blankets to reduce the amount of seepage or downstream horizontal drainage blankets for stability against seepage forces. Weak foundations may require the addition of stabilizing fills at either or both toes of the dam. The additional embankments needed because of pervious or weak foundations should be provided beyond the slopes determined herein as required for embankment stability.

The upstream slope may vary from 2 : 1 to as flat as 4 : 1 for stability; usually it is 2½ : 1 or 3 : 1. Flat upstream slopes are sometimes used in order to eliminate expensive slope protection. A berm is often provided at an elevation slightly below the maximum drawdown of the reservoir water surface to form a base for the upstream slope protection, which need not be carried below this point. The upstream slope is often steepened above the elevation where water is stored; that is, in the surcharge range.

A storage dam subject to rapid drawdown of the reservoir should have an upstream zone with permeability sufficient to dissipate pore-water pressures exerted outwardly in the upstream part of the dam. The rate of reservoir drawdown is an important factor which affects the stability of the upstream part of the dam. Where only fine material of low permeability is available, such as that predominating in clays, it is necessary to provide a flat slope if rapid drawdown is a design requirement. Conversely, if free-draining sand and gravel are available to provide a superimposed weight for holding down the fine material of low permeability, a steeper slope may be used. The same result may be secured by utilizing sound and durable rock from required excavations. In the latter case, a layer of sand and gravel or quarry fines must be placed between the superimposed rock and the surface of the impervious embankment to prevent damage and displacement from saturation and wave action.

Flood damage due to failure of the upstream face is very unlikely. Failure can take place only during construction or following a rapid drawdown; in both cases the reservoir should be virtually empty. The weight and seepage forces act as a stabilizing influence on the upstream face when the reservoir is full.



EXCAVATION

ITEMS	DESCRIPTION	QUANTITY CU YDS
1	Exc for O W works	8,900
2	Exc for emergency spillway	5,900
3	Exc for dam emb foundation	17,000
4	Exc for stripping borrow pits	2,000
5	Exc borrow area separation and transportation to dam embankment	48,000
	Total	79,800

EMBANKMENT

ITEMS	DESCRIPTION	QUANTITY CU YDS
6	Pervious backfill	225
7	Bedding for riprap	200
8	Earth fill in dam embankment Zone 1	45,000
9	Specially compacted earth fill	200
10	Cobble & rock fill in dam emb. Zone 2	9000
11	Riprap	2,700
	Total	58,325

Figure 119. Materials distribution chart for Wasco Dam, an earthfill structure on Clear Creek in Oregon.

484556 O—60——15

The usual downstream slopes for small earthfill dams are 2 : 1 where a downstream pervious zone is provided in the embankment, and 2½ : 1 where the embankment is impervious. These slopes are stable for soil types commonly used when drainage is provided in the design so that the downstream slope of the embankment does not become saturated by seepage.

The slopes of an earthfill dam depend on the type of dam (that is, diaphragm, modified homogeneous, or zoned embankment), and on the nature of the materials for construction. Of special importance is the nature of the soil which will be used for construction of the modified homogeneous dam or the core of a zoned dam. In the latter case, the relation of the size of the core to the size of the shell is also significant.

In this text, the slopes of the embankment are related to the classification of the soil to be used for construction, especially the impervious soils. The engineering properties of soils in the various classifications are shown in table 6 (sec. 89). The slopes chosen are necessarily conservative and are recommended only for small earthfill dams within the scope of this text, as discussed in section 119.

(c) *Diaphragm Type.*—A diaphragm type of earthfill dam is recommended for small dams only where the supply of impervious soil is so limited that the zoned embankment type cannot be constructed. In that event, it is recommended (sec. 120) that for small dams a diaphragm of manufactured material be placed on the upstream slope of an otherwise pervious embankment in lieu of a soil blanket. If the pervious material is rock, the dam is classified as a rockfill dam, the design of which is discussed in chapter VI.

The pervious material for construction of a diaphragm earthfill dam must be such that it can be compacted to form a stable embankment which will be subject to only small amounts of postconstruction settlement. Poorly graded sands cannot be satisfactorily compacted; well-graded sand-gravel mixtures or well-graded gravels make satisfactory embankments. Well-graded sand-gravel mixtures which contain more than 5 percent of material finer than the 200-mesh sieve should be tested to determine that they will form free-draining embankments after compaction. Well-compacted pervious embankments are very stable, and 2 : 1 slopes both upstream and downstream are adequate for small dams. Although steeper slopes likely would be stable, they are not economical to construct.

In all respects, except for the use of pervious materials other than rock in construction of the embankment, the diaphragm earthfill dam design as recommended herein for small dams is identical with the design of rockfill dams, as discussed in chapter VI. That discussion should be referred to for the design of foundations and upstream facings for a diaphragm-type earthfill dam.

(d) *Homogeneous Type.*—The homogeneous type of dam is recommended only where the paucity of free-draining materials makes the construction of a zoned embankment type uneconomical, and with the further qualification that for storage dams the homogeneous dam must be modified to provide for the inclusion of internal drainage facilities. The recommended drainage facilities are described in section 120 and are shown on figure 96. If the rockfill toe shown on figure 96(A) is provided, a filter must be constructed between the embankment proper and the rockfill. This filter and the filter drain shown in figure 96(B) should be designed as described in section 126(h).

To perform its function of lowering the phreatic line and stabilizing the downstream portion of the dam, the filter drain shown on figure 96(B) should extend from the downstream slope of the dam to well within the body of the embankment. However, it should not extend upstream so far as to reduce the length of the path of percolation through the embankment or the foundation to a dangerous extent. Also, a minimum-length filter drain is desirable because filters are expensive to construct. For small dams, it is recommended that the filter drain start at the downstream toe of the embankment and extend upstream to within a distance equal to the height of the dam plus 5 feet from the centerline of the dam. This will afford a drain of ample extent and yet not reduce the length of the path of percolation beyond desirable limits. The distance of height of dam plus 5 feet is selected on the basis that this will place the upstream limit of the filter drain at a position corresponding to the upstream limit of a pervious zone for a zoned embankment constructed on a pervious foundation without a positive cutoff trench, as shown on figure 120.

The filter drain should be carried across the valley floor and up the abutments to an elevation

corresponding to the highest level at which water will be stored in the reservoir for an appreciable time. It should be a uniformly thick blanket whose upstream position at a given point is downstream from the centerline of the dam a distance equal to the height of the dam at that point plus 5 feet.

Even in the construction of a homogeneous embankment, there is likely to be some variation in the nature of the borrow material. It is important that the coarser and more pervious of the materials available be placed at the outer slopes in order to approach, as nearly as possible, the advantages of a zoned embankment.

The recommended slopes for small homogeneous earthfill dams are shown in table 15 for detention dams and for storage dams with and without rapid drawdown as a design condition. Where more than one soil classification is shown for a set of slopes, it indicates that the dam can be constructed to these slopes using any one of these soils or any combination thereof.

(e) *Zoned Embankment.*—Section 120 describes the zoned embankment dam and states that this type of dam should always be constructed where there is a variety of soils readily available because its inherent advantages will lead to economies in the cost of construction. This type of design is economical to construct because it permits the use of steeper slopes with a consequent reduction in the total volume of embankment material and because it also allows a wide variety of material to be used. It thus allows for maximum utilization of material which must be excavated for the foundations of the dam, spillway, and outlet works.

The scheme of zoning may divide the dam into three or more sections, depending on the range of variation in the character and gradation of the available materials of construction. Relatively free-draining materials, and therefore, those with a high degree of inherent stability are used to enclose and support the less stable impervious core. Pervious materials are placed in the downstream sections to avoid building up of pressures from percolating water and to permit lowering the phreatic line so as to keep it well within the toe of the embankment. Pervious materials are placed in upstream sections to permit dissipation of pressures on rapid drawdown. When there is a great enough variation in available material, several zones may be used. Beginning at the impervious section, each zone is of increasingly permeable materials toward the outer slope.

It is important that the gradation of adjacent zones be considered so that materials from one zone are not piped into the voids of adjoining zones, either by steady-stage or by drawdown seepage forces. A transition of sand-gravel or rock fines must be provided between an impervious zone and a flanking rockfill. If the transition zone is only a few feet thick, it should be designed as a filter in accordance with the procedure given in section 126(h). Transition zones ordinarily are not required between impervious and sand-gravel zones or between sand-gravel zones and rockfill.

The slopes required for stability of a zoned embankment are a function of the relative sizes of the impervious core and the pervious flanking zones. Figure 120 shows the "minimum" core for a dam constructed on an impervious foundation or on a pervious foundation which is com-

TABLE 15.—*Recommended slopes for small homogeneous earthfill dams on stable foundations*

Case	Type	Purpose	Subject to rapid drawdown [1]	Soil classification [2]	Upstream slope	Downstream slope
A	Homogeneous or modified-homogeneous	Detention or storage	No	GW, GP, SW, SP	Pervious, not suitable	
				GC, GM, SC, SM	2½:1	2:1
				CL, ML	3:1	2½:1
				CH, MH	3½:1	2½:1
B	Modified-homogeneous	Storage	Yes	GW, GP, SW, SP	Pervious, not suitable	
				GC, GM, SC, SM	3:1	2:1
				CL, ML	3½:1	2½:1
				CH, MH	4:1	2½:1

[1] Drawdown rates of 6 inches or more per day following prolonged storage at high reservoir levels.

[2] OL and OH soils are not recommended for major portions of homogeneous earthfill dams. Pt soils are unsuitable.

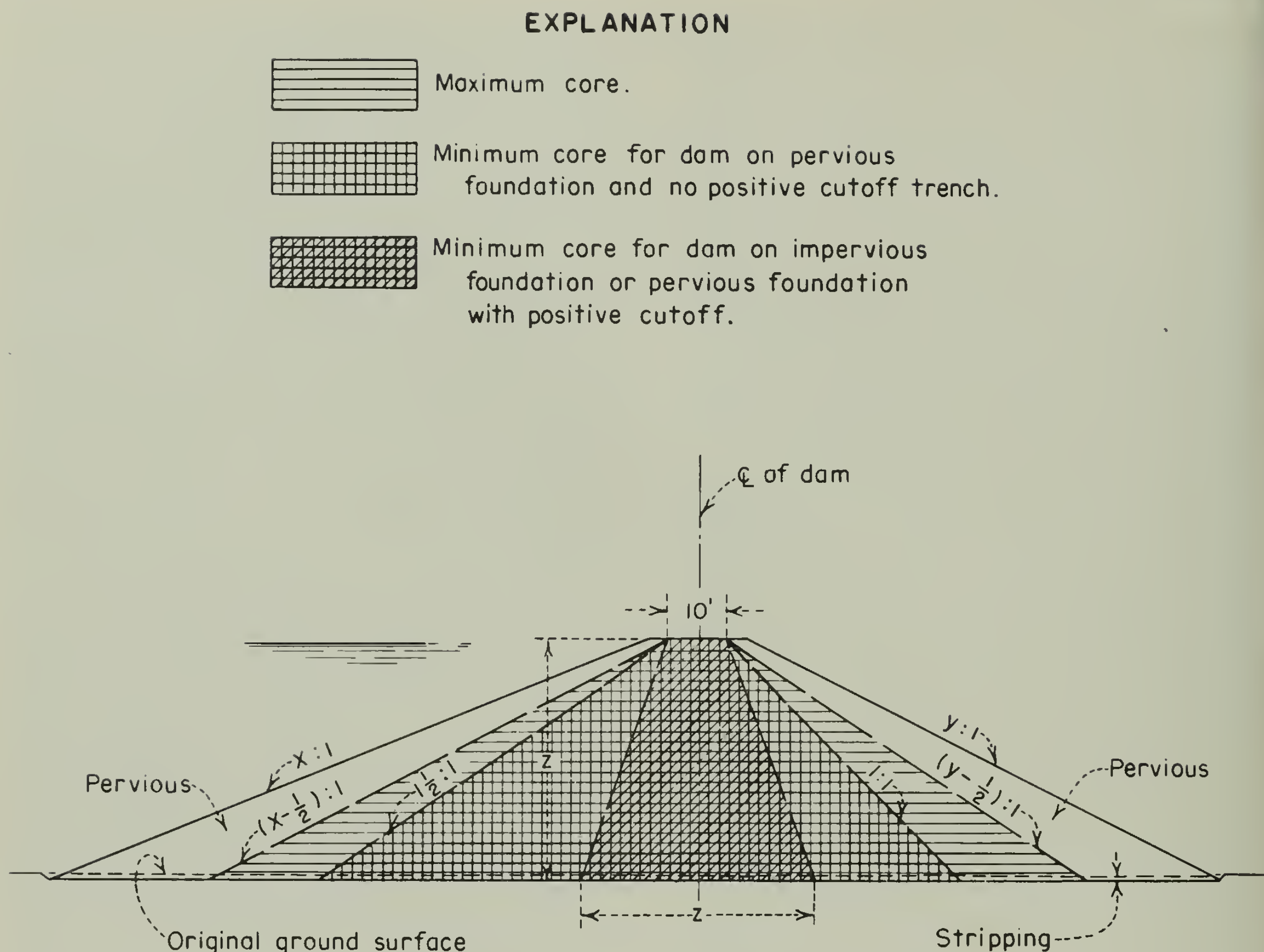


Figure 120. **Size range of impervious cores in zoned embankment.**

pletely penetrated by a cutoff trench (hereinafter referred to as the minimum core); the minimum core for a dam constructed on a pervious foundation which is not completely penetrated by a cutoff trench (hereinafter referred to as the minimum core for pervious foundations); and the "maximum" size core for a zoned-type dam. If the core is smaller than the minimum size shown, the dam is considered to be of the diaphragm type; if the core is larger than the maximum size shown, the pervious zones are largely ineffectual in stabilizing the core, and the embankment may be considered as the homogeneous type so far as stability is concerned.

The size of the minimum core shown in figure 120 was selected for both practical and theoretical reasons. The width of 10 feet was taken as the minimum that will permit economical placement and compaction of impervious embankment material by construction equipment, which may include trucks, dozers, and tamping rollers. The criterion that the thickness of the core at any elevation can not be less than the height of embankment at that elevation was adopted so that the average hydraulic gradient through the core will be less than unity. Appreciably steeper gradients result in high seepage forces and the necessity for construction of high-quality filter zones, which for small dams are neither economical nor practicable from a construction control standpoint. Furthermore, if the core were thinner there would be more danger that it could be ruptured by differential settlement of the foundation.

The minimum core for a dam on a pervious foundation, as shown on figure 120, is based on consideration of seepage pressures in the foundation. This minimum applies to dams constructed on pervious foundations which are exposed (case

1). It does not apply if the pervious foundations are mantled by an impervious layer (case 2) more than 3 feet thick. (Refer to sec. 127(a) for a discussion of the several cases of pervious foundations.) If a cutoff trench completely penetrating the case 1 pervious foundation is not provided in the design, it must be anticipated, regardless of what other type of device is utilized to control seepage, that the loss of head through the foundation will be relatively gradual and proportional to the length of the seepage path. The minimum length of path used in practice in conjunction with seepage control devices as shown on figure 107(B), 107(C), or figure 108 is that provided by an impervious zone whose thickness at the contact of the dam with the foundation is at least 2½ times the height of the dam. To conform to good practice and to avoid the possibility of seepage passing under the core of the dam without an appreciable loss of head because of the ineffectiveness of sheet piling or a partial cutoff trench, or because no such device is provided, it is recommended that the minimum size impervious zone shown on figure 120 for a dam on a pervious foundation be used on all case 1 pervious foundations for which positive cutoff trenches are not provided.

With the minimum size core centrally located as shown in figure 120, the stability of the zoned embankment is not greatly affected by the nature of the soil comprising the core. The outside slopes are governed largely by the stability of the shell material. Rock, well-graded gravels (GW), and poorly graded gravels (GP) provide suitable material for the shell. Well-graded sand (SW) and poorly graded sand (SP) are suitable if they are gravelly. For any of these materials, upstream and downstream slopes of 2 : 1 are stable for dams not exceeding 50 feet in height above the lowest point in the streambed, even if subject to rapid drawdown.

Table 16 shows the recommended slopes for zoned small earthfill dams with minimum and maximum cores. Slopes of zoned small earthfill dams with cores of intermediate size (including the minimum core for a dam on a pervious foundation) will fall between those given in the table for case A and for the appropriate case with maximum size core. Where more than one soil classification is shown for a set of slopes, it indicates that the dam can be constructed to these slopes using any one of these soils, or any combination of them.

TABLE 16.—*Recommended slopes for small zoned earthfill dams on stable foundations*

Case	Type	Purpose	Subject to rapid drawdown [2]	Shell material classification	Core material classification [3]	Upstream slope	Downstream slope
A	Zoned with "minimum" core.[1]	Any	Not critical [4]	Not critical; Rockfill, GW, GP, SW (gravelly), or SP (gravelly).	Not critical, GC, GM, SC, SM, CL, ML, CH, or MH.	2:1	2:1
B	Zoned with "maximum" core.[1]	Detention or storage.	No	Not critical; Rockfill, GW, GP, SW (gravelly), or SP (gravelly).	GC, GM	2:1	2:1
					SC, SM	2¼:1	2¼:1
					CL, ML	2½:1	2½:1
					CH, MH	3:1	3:1
C	Zoned with "maximum" core.[1]	Storage	Yes	Not critical; Rockfill, GW, GP, SW (gravelly), or SP (gravelly).	GC, GM	2½:1	2:1
					SC, SM	2½:1	2¼:1
					CL, ML	3:1	2½:1
					CH, MH	3½:1	3:1

[1] "Minimum" and "maximum" size cores are as shown on fig. 120.
[2] "Rapid" drawdown is a drawdown rate of 6 inches or more per day following prolonged storage at high reservoir levels.
[3] OL and OH soils are not recommended for major portions of the cores of earthfill dams. Pt soils are unsuitable.
[4] Rapid drawdown will not affect the upstream slope of a zoned embankment which has a large upstream pervious shell.

E. EMBANKMENT DETAILS

135. ***Crest Design.***—The crest width of an earthfill dam depends on several considerations such as: (1) nature of embankment materials and minimum allowable percolation distance through the embankment at normal reservoir water level, (2) height and importance of structure, (3) possible roadway requirements, and (4) practicability of construction. A minimum crest width should be

that width which will provide a safe percolation gradient through the embankment at the level of the full reservoir. Because of practical difficulties in determining this factor, the crest width is, as a rule, determined empirically and largely by precedent. The following formula is suggested for the determination of crest width for small earthfill dams:

$$w=\frac{z}{5}+10 \qquad (12)$$

where:

w=width of crest in feet, and

z=height of dam in feet above lowest point in the streambed.

For ease of construction with power equipment, the minimum width should not be less than 12 feet. In certain instances the minimum width may be determined by the requirement for a roadway across the dam.

Some type of surfacing should be placed on the crest for protection against damage by wave splash and spray, rainfall runoff and wind, and traffic wear and tear when the crest is used as a roadway. The usual treatment consists of placing a layer of selected fine rock or gravelly material to a minimum thickness of 4 inches. In the event the crest constitutes a section of a highway, the width of roadway and kind of surfacing should conform to those of the highway with which it connects. Surface drainage of the crest should be provided by a crown of at least 3 inches, or by sloping the crest to drain towards the upstream slope. The latter method is preferred unless the downstream slope is protected against erosion by surfacing as resistant as that afforded by the protection on the upstream slope.

If the crest of the dam is to provide for a highway, cable or beam-type guard rails are usually constructed along both shoulders of the crest. If a highway crossing is not anticipated, the crest can be delineated by posts at 25-foot intervals or by boulders placed at intervals along the crest, although in many instances treatment is not required.

Suitable parking areas should be provided for the convenience of visitors and others at the abutments of the dam, especially for a storage dam whose lake will be used for recreational purposes. Consideration should be given to providing a turnaround where vehicle traffic is permitted to reach one end of a crest which dead ends into the opposite abutment.

Camber is ordinarily provided along the crest of earthfill dams to insure that the freeboard will not be diminished by foundation settlement or embankment consolidation. Selection of amount of camber is necessarily somewhat arbitrary; it is based on the amount of foundation settlement and embankment consolidation expected for a particular dam, with the objective of providing enough so that some residual camber will remain after settlement and consolidation. Impervious embankment materials placed at densities roughly corresponding to the Proctor laboratory maximum will consolidate appreciably when subject to overlying fill loads. It is expected, however, that the major portion of this consolidation will take place during construction before the embankment is completed, and therefore the expected foundation settlement is the more important factor. For dams on relatively noncompressible foundations, cambers of about 1 percent of the height are commonly provided. Several feet of camber may be required for dams constructed on foundations which may be expected to settle. Parabolic or straight-line equations may be used to vary the amount of camber and to make it roughly proportional to the height of the embankment.

The additional amount of embankment material required to provide camber in the crest of an embankment is nominal, as the increased height to the embankment is provided by pitching the slopes near the crest of the dam. The modifications to the section of the embankment due to the addition of camber are not taken into account in selecting slopes for stability.

The crest details on figure 154 show how 6 inches of camber was provided for a 46-foot-high Bureau of Reclamation dam (Wasco Dam), constructed on a firm foundation, by steepening the embankment slopes near the top of the dam. If more camber had been considered necessary, the point of intersection of the steepened slopes and the normal slopes would have been lowered to avoid oversteepening. Figure 154 also shows the profile on the centerline of the same dam, and the camber diagram which varied the amount of camber proportionately to the height of the dam above the bottom of the cutoff trench excavation.

The crest details on figure 154 also illustrate modifications to the zone lines which are ordinarily

made near the crests of zoned embankments in order to avoid zones of extremely narrow width. In this case the impervious zone was narrowed near the top of the dam to avoid narrow pervious zones which would be difficult to construct. A minimum top width of 14 feet was maintained for the impervious zone to insure adequate room for compaction by tamping rollers.

Another common modification to impervious zones of earthfill dams consists of ending the zone below the crest of the dam, as shown in the crest details on figure 154. This is done not only to facilitate construction, but also for other reasons. Impervious zones extending to the top of the dam are subject to damage by frost action, which causes loosening of the soil, and to the formation of unsightly shrinkage cracks upon drying out of the soil. However, the top of the impervious core must be maintained several feet above the maximum water surface to prevent excessive percolation through the embankment when the reservoir is full. In the example, the maximum water surface elevation is 3520 feet above sea level, while the high point of the zone 1 is at elevation 3523.5. Note that the top of the zone 1 is also sloped to facilitate drainage.

136. *Freeboard.* Freeboard is the vertical distance between the crest of the embankment (without camber) and the reservoir water surface. The more specific term "normal freeboard" is defined as the difference in elevation between the crest of the dam and the normal reservoir water level as fixed by design requirements. The term "minimum freeboard" is defined as the difference in elevation between the crest of the dam and the maximum reservoir water surface that would result should the inflow design flood occur and should the outlet works and spillway function as planned. The difference between normal and minimum freeboard represents the surcharge head (sec. 181). If the spillway is uncontrolled, there will always be a surcharge head; if the spillway is gated, it is possible for the normal and minimum freeboards to be identical.

A distinction is made between normal and minimum freeboards because of the different requirements for freeboard if surcharge head is involved. The normal freeboard must meet the requirements for longtime storage. It must be sufficient to prevent seepage through a core which has been loosened by frost action or which has cracked due to drying out. This is of particular importance for a dam whose core is a CL or CH material when located in areas where either very cold or very hot dry climates are encountered. It must also be sufficient to prevent overtopping of the embankment by abnormal and severe wave action of rare occurrence that may result from unusual sustained winds of high velocity from a critical direction.

Minimum freeboard is provided to prevent overtopping of the embankment by wave action which may coincide with the occurrence of the inflow design flood. Minimum freeboard also provides a safety factor against many contingencies such as settlement of the dam more than the amount anticipated in selecting the camber, occurrence of an inflow flood somewhat larger than the inflow design flood, or malfunction of spillway controls or outlet works with an increase in maximum water surface above that expected. In some instances, especially where the maximum probable inflow is used as a basis for design, the minimum freeboard may be established on the assumption that the dam should not be overtopped as a result of malfunction of the controlled spillway or outlet works which would result from human or mechanical failure to open gates or valves. In such instances, allowances for wave action or other contingencies usually are not made.

The rational determination of freeboard would require a determination of the height and action of waves. The height of waves generated by winds in a reservoir depends on the wind velocity, the duration of the wind, the fetch,[3] depth of water, and the width of the reservoir. The height of the waves as they approach the upstream face of the dam may be altered by the increasing depth of water, or by the decreasing width of reservoir. Upon contact with the face of the dam, the effect of waves is influenced by the angle of the wave train with the dam, the slope of the upstream face, and the texture of the slope surface. The sloping face of an earthfill dam allows the waves to move up the inclined plane and expend part of their energy in raising the water instead of in direct force upon the face itself, as against a vertical wall. The rough surface of dumped riprap re-

[3] Fetch is the distance over which the wind can act on a body of water. It is generally defined as the normal distance from the windward shore to the structure being designed, although the "effective" fetch may have a slightly curved path, as in the case of the wind sweeping down a winding river valley between land ridges.

duces wave uprush to approximately 1.5 times the height of the wave, but uprush may be considerably more for smooth surfaces such as concrete. Because there are no specific data on wave height and wave rideup the determination of freeboard requires judgment and consideration of local factors.

A summary of empirical formulas proposed for determination of wave heights is given in an American Society of Civil Engineers report [28], from which the following table was extracted:

Fetch, miles	Wind velocity, miles per hour	Wave height, feet
1	50	2.7
1	75	3.0
2.5	50	3.2
2.5	75	3.6
2.5	100	3.9
5	50	3.7
5	75	4.3
5	100	4.8
10	50	4.5
10	75	5.4
10	100	6.1

All conditions affecting exposure of the dam to the wind must be considered in selecting the maximum wind velocity. It is believed that from a geographical standpoint no locality is safe from an occurrence of winds of up to 100 miles per hour at least once during a period of many years, although a particular site may be topographically sheltered so that the reservoir is protected from sustained winds of high velocity. Under these conditions velocities of 75 or even 50 miles per hour may be used.

For the design of small dams with riprapped slopes, it is recommended that the freeboard be sufficient to prevent overtopping of the dam due to wave rideup equal to 1.5 times the height of the wave as interpolated from the above tabulation, measured vertically from the still water level. Normal freeboard should be based on a wind velocity of 100 miles per hour, and minimum freeboard on a velocity of 50 miles per hour. Based on these assumptions and on other considerations of the purpose of freeboard, as previously discussed, the following tabulation lists the least amount recommended for both normal and minimum freeboard on riprapped earthfill dams; the design of the dam should satisfy the most critical requirement.

Fetch, miles	Normal freeboard, feet	Minimum freeboard, feet
Less than 1	4	3
1	5	4
2.5	6	5
5	8	6
10	10	7

An increase in the freeboard shown above for dams where the fetch is 2.5 miles and less may be required if the dam is located in very cold or in very hot dry climates, particularly if CL and CH soils are used for construction of the cores. It is also recommended that the amount of freeboard shown in the tabulation be increased by 50 percent if a smooth pavement is to be provided on the upstream slope.

137. *Upstream Slope Protection.*—(a) *General.*—The upstream slopes of earthfill dams must be protected against destructive wave action. In some instances, provision must be made against burrowing animals. Usual types of surface protection for the upstream slope are rock riprap, either dry-dumped or hand-placed, and concrete pavement. Other types of protection that have been used are steel facing, bituminous pavement, precast-concrete blocks, and (on small and relatively unimportant structures) willow mattresses and sacked concrete. The upstream slope protection should extend from the crest of the dam to a safe distance below minimum water level (usually several feet) and ordinarily should terminate on a supporting berm.

(b) *Selection of Type of Protection.*—Experience has shown that in the majority of cases dumped riprap furnishes the best type of upstream slope protection at the lowest ultimate cost. Approximately 100 dams, located in various sections of the United States with a wide variety of climatic conditions and wave severity, were examined by the U.S. Corps of Engineers as a basis for establishing the most practical and economical means for slope protection [29]. The dams were from 5 to 50 years old and were constructed by various agencies. This survey found that:

(1) Dumped riprap failed in 5 percent of the cases where used, failures being attributed to improper size of stones.

(2) Hand-placed riprap failed in 30 percent of the cases where used, due to the usual method of single-course construction.

(3) Concrete pavement failed in 36 percent of the cases where used, due generally to the inherent deficiencies of this type of construction.

This survey substantiated the premise that dumped riprap is by far the preferable type of upstream slope protection. The excellent service rendered by dumped riprap is exemplified in the case of Cold Springs Dam, constructed by the Bureau of Reclamation. Figure 121 shows the condition of the riprap on the upstream slope of this dam after 50 years of service. The only maintenance required during that period has been the replacement of some riprap which was dislodged near the center of the dam by a particularly severe storm in 1931. Although some beaching action has taken place subsequently, it has not been severe enough to require further maintenance.

The superiority of dumped rock riprap for upstream slope protection and its low cost of maintenance compared to other types of protection have been demonstrated so convincingly that it has been considered economical to import rock from considerable distances to avoid construction of other types of slope protection for major dams. For example, the Bureau of Reclamation has imported rock from sources which required a rail haul of over 200 miles and a truck haul of 24 miles from the railhead to the dam, and the Corps of Engineers has imported rock from a distance of 170 miles.

When the nearest source of suitable rock is located far from the site, and especially when only small quantities are involved, it may be economical to use hand-placed riprap despite its higher unit cost for labor and material because a lesser thickness may be used. Hand-placed riprap is satisfactory where not exposed to heavy ice conditions, but the rock must be of better quality than the minimum suitable for dumped riprap, and placement must be such that the hand-placed riprap approaches good dry rubble in quality and appearance. It should be recognized that hand-placed riprap is not as flexible as dumped riprap, since it cannot adjust itself as well to foundation or local settlements. Consequently, hand-placed riprap should not be used where considerable settlement is expected.

Concrete paving deserves serious consideration for upstream slope protection where the use of riprap is too expensive because of high transportation costs. The success of concrete pavement as a slope protection medium depends on the evaluation of field conditions and the assumptions made as to the behavior of the embankment and the ability of the paving to resist cracking and deterioration. Concrete pavement has proved to be satisfactory in some cases under moderate wave action. An example is at McKay Dam, constructed by the Bureau of Reclamation near Pendleton, Oreg. This pavement, although exposed to severe weather conditions, is in excellent condition after more than 30 years of service, as shown in figure 122.

Figure 121. Riprap on upstream slope of an earthfill dam in excellent condition after 50 years of service. The structure is Cold Springs Dam, which forms an offstream reservoir on the Umatilla project in Oregon. IO–2194.

Figure 122. Paved upstream slope of an earthfill dam in excellent condition after 30 years of service. The structure is McKay Dam on a tributary of the Umatilla River in Oregon. IO–2198.

Where severe wave action is anticipated, concrete pavement appears practicable only when the settlement within the embankment after construction will be insignificant. In comparing the cost of concrete pavement and riprap, the cost of any additional foundation measures necessary to minimize settlement and the additional freeboard required because of greater wave rideup on the smooth surface should be considered.

Other types of upstream slope protection, such as precast concrete blocks, bituminous paving, soil-cement, and steel facing, should be considered only in very unusual circumstances. Bituminous paving and soil-cement paving are experimental types. Precast concrete blocks and steel facing are types which are usually not competitive from a cost standpoint and which, in general, have poor service histories. Willow mattresses and sacked concrete should be used only on very minor structures, and then only when the cost of a more permanent type of slope protection is prohibitive.

For purposes of this text, the designs of the following types of slope protection are discussed: Dumped rock riprap, hand-placed rock riprap, concrete pavement, and brush mattresses.

(c) *Dumped Rock Riprap.*— Dumped rock riprap consists of stones or rock fragments dumped in place on the upstream slope of an embankment to protect it from wave action. The riprap is placed on a properly graded filter which may be a specially placed blanket or may be the upstream zone of a zoned embankment. Figure 123 shows riprap being placed on Bonny Dam, constructed by the Bureau of Reclamation. This riprap is composed of angular granite gneiss rock of high specific gravity and of excellent quality.

Figure 123. **Placing riprap on upstream slope of Bonny Dam. Riprap was placed on graded filter visible in foreground. Bonny Dam is on the South Fork of the Republican River in Colorado.**

The efficacy of dumped rock riprap depends on the following characteristics:

(1) Quality of the rock.

(2) Weight or size of the individual pieces.

(3) Thickness of the riprap.

(4) Shape of the stones or rock fragments.

(5) Slopes of the embankment on which the riprap is placed.

(6) Stability and effectiveness of the filter on which the riprap is placed.

Rock for riprap should be hard, dense, and durable, and should be able to resist long exposure to weathering. Most of the igneous and metamorphic rocks, many of the limestones, and some of the sandstones make excellent riprap. Limestones and sandstones that have shale seams are undesirable. The suitability of rock for riprap from a quality standpoint is determined by visual inspection, by laboratory tests to determine the resistance to weathering and to abrasion, and by petrographic examination to determine the structure of the rock as it affects its durability. The laboratory tests are described in chapter IV.

The individual pieces must be of sufficient weight to resist displacement by wave action, which is not necessarily a function of the height of the dam. It is a misconception to consider that large-size rocks are needed only on higher structures, while small-size rocks will afford ample slope protection for low fills, without regard to factors such as wind velocity, wind direction, and fetch distance. This can be demonstrated by comparing figure 124 with figure 121. Cold Springs Dam (fig. 121) is a 90-foot-high dam whose upstream slope is protected by a 24-inch layer of basalt rock with the average weight of the larger fragments probably not exceeding 100 pounds. The wave action on this reservoir is not severe, and the riprap has given satisfactory service for 50 years with relatively little maintenance required. Figure 124 shows riprap containing relatively large fragments that has been dislodged from the upstream slope of a low dike section of another dam subject to heavier wave action.

The weight or size of the individual pieces required to resist displacement by wave action may be determined theoretically by the methods given in the American Society of Civil Engineers report referred to in the discussion of freeboard require-

Figure 124. **Displacement of riprap on a low dike by wave action.**

ments [28]. This method is based on the premise that the force a wave exerts on riprap stones on the face of a dam cannot be greater than that of a current flowing at a velocity equal to the velocity of the water particles of the wave. These theoretical methods are in good agreement with the experience and analysis of results obtained on a large number of earthfill dams by the Bureau of Reclamation.

The thickness of the riprap should be sufficient to accommodate the weight and size of stone necessary to resist wave action. The Bureau of Reclamation has found a 3-foot thickness of dumped riprap to be generally most economical and satisfactory for major dams. Lesser thicknesses are used on low dams or on dike sections where wave action will be less severe than on principal structures. Less thickness also has been specified for the upper slopes of dams whose reservoirs are largely allocated to flood control, because of the infrequent and short periods of time that the upper slopes are subject to wave action. Greater thicknesses have been specified in cases where rock having a low specific gravity (less than 2.50) was used. Table 17 shows the recommended thickness and gradation of dumped rock riprap for small dams for various fetches, based on theoretical considerations and the experience and practice of the Bureau of Reclamation.

The shape of the individual stones or rock fragments influences the ability of the riprap to resist displacement by wave action. Angular fragments of quarried rock tend to interlock and resist displacement better than do boulders and

TABLE 17.—*Thickness and gradation limits of riprap on 3:1 slopes*

Reservoir fetch, miles	Nominal thickness, inches	Gradation, percentage of stones of various weights, pounds			
		Maximum size	25 percent greater than	45 to 75 percent from to—	25 percent less than [1]
1 and less	18	1,000	300	10–300	10
2.5	24	1,500	600	30–600	30
5	30	2,500	1,000	50–1,000	50
10	36	5,000	2,000	100–2,000	100

[1] Sand and rock dust less than 5 percent

rounded cobbles. The values given in table 17 are for angular quarried rock. If boulders or rounded cobbles are to be used such as shown in figure 19 a thicker layer containing larger sizes may be required, or the slope of the embankment may need to be made flatter than required for stability in order for the boulder and cobble riprap to stay in place, especially if cobbles of relatively uniform diameter are to be used.

Table 17 is for riprap thickness and gradation on 3 : 1 slopes. For 2 : 1 slopes, the nominal thickness required (except the 36-inch thickness) should be increased by 6 inches and the corresponding gradation used.

A layer or blanket of graded gravel should be provided underneath the riprap when the compacted material of the underlying earthfill is of such gradation that there is danger that fines may be washed out through the voids in the riprap by wave action, resulting in undermining of the riprap. A blanket is usually not required if the outer zone of a zoned embankment is gravel. Blankets of crushed rock or natural gravel graded from $^3/_{16}$ to $3^1/_2$ inches with a thickness equal to one-half the thickness of the riprap (but not less than 12 inches) have proved satisfactory in practice. The blanket gradation may be determined more exactly by the filter criteria given in section 126(h).

(d) *Hand-Placed Rock Riprap.*—A good example of hand-placed rock riprap is shown in figure 125. The upstream slope protection is in an excellent state of preservation after 36 years of service. Hand-placed riprap consists of stones carefully laid by hand in a more or less definite pattern with a minimum amount of voids and with the top surface relatively smooth. Rounded or irregular stones lay up less satisfactorily and rapidly than stone which is roughly square; stone of a

flat, stratified nature should be placed with the principal bedding planes normal to the slope. Joints should be broken as much as possible, and joint openings to the underlying fill should be avoided by carefully arranging the various sizes of stones and closing the openings with spalls or small rock fragments. However, there should be enough openings in the surface of the riprap to vent the subsurface properly.

Stone for hand-placed rock riprap must be of excellent quality. The thickness of hand-placed rock riprap should be one-half of the thickness required for dumped rock riprap but not less than 12 inches, and a filter blanket should be provided underneath the riprap if the underlying zone of the earthfill dam is not gravel.

Figure 125. Hand-placed rock riprap on Indian Creek dike, an offstream dike for Strawberry Reservoir, part of a transmountain diversion project in Utah.

(e) *Concrete Paving.*—If a complete history could be gathered concerning the numerous instances where concrete paving was used for the protection of the upstream slopes of small dams, it would be found that the number of failures is tremendous. However, the fact that some structures protected with concrete facings have withstood the test of time continues to lead engineers to use this type of construction, often without sufficient reference to other past performance records. A properly designed and constructed concrete facing is never cheap. The uncertainty and complexity of the forces which may act on a concrete paving makes it desirable to use conservative treatment whenever this type of slope protection is considered. The recommendations that follow should provide the necessary degree of conservatism, but the number of situations studied is so limited that there is no assurance that adequate consideration has been given to every type of hazard that may be encountered.

Concrete paving used for slope protection should extend from the crest of the dam to several feet below the minimum water surface. It should terminate on a berm and against a concrete curb or header which should extend at least 18 inches below the undersurface of the paving.

For dams approaching 50 feet in height, a paving thickness of 8 inches is recommended; the minimum thickness for lower dams should be 6 inches. Although concrete paving has been constructed in blocks, the generally favored method which has given the best service is to make the paving monolithic to the greatest extent possible, with every measure taken to prevent access of water and consequent development of hydrostatic pressures underneath the concrete. The good service given by the concrete pavement on the upstream slope of McKay Dam (fig. 122) is attributed to monolithic type of construction, durability of concrete, little settlement of the dam or foundation, and pervious nature of underlying fill which prevents development of hydrostatic uplift pressures even though a minor amount of cracking has occurred.

In contrast with the success of concrete paving at McKay Dam is the experience of the Bureau of Reclamation with the concrete paving at Belle Fourche Dam. In this instance monolithic construction was not used. The paving consists of 8-inch-thick blocks, 6 feet 6 inches by 5 feet, placed directly upon the impervious underlying embankment. The condition of the paving after 40 years of service is shown in figure 126. Considerable maintenance of the paving has been required through the years; a number of the blocks have been displaced and broken up by wave action and uplift forces under the slabs. Compared to the general service record of riprap or the concrete pavement in figure 122, this slope protection design cannot be deemed successful.

If monolithic construction is not possible, expansion joints should be kept to a minimum and construction joints should be spaced as widely as possible. The slab should be reinforced with bars in both directions, placed at middepth of the slab, and made continuous through the construction joints. An area of steel in each

direction equal to 0.5 percent of the area of the concrete is considered good practice. Joints should be sealed with plastic fillers, and subsequent open cracks in the concrete should be grouted or sealed promptly.

(f) *Brush Mattresses.*—Brush mattresses, usually of willow, tamarisk, or cottonwood, when employed on small dams as a substitute for more costly surface protection, are made of saplings 1 to 2 inches in diameter and as long as 20 feet, assembled in bundles 12 to 18 inches in diameter and tied with wire. The bundles are laid on the face of the dam, lengthwise of the slope with butts downhill, and are then woven together with heavy wire or cable. The cable is anchored to stout

Figure 126. **Concrete paving blocks on the upstream slope of Belle Fourche Dam. Note deteriorated condition of this type of paving after 40 years' service in spite of considerable maintenance. Belle Fourche Dam is an earthfill structure on a tributary of the Belle Fourche River in South Dakota.**

posts set deep into the embankment or preferably to concrete anchor blocks. Tie cables should be not over 3 feet apart, and closer if a single-strand heavy wire is used. The mat should have a minimum thickness of 1 foot for small dams and 1½ to 2 feet for larger structures A fundamental objection to brush mats is the need for frequent replacement.

138. ***Downstream Slope Protection.***—If the downstream zone of an embankment consists of rock or cobble fill, no special surface treatment of the slope is necessary. Downstream slopes of homogeneous dams or dams with outer sand and gravel zones should be protected against erosion by wind and rainfall runoff by a layer of rock, cobbles, or sod. Because of the uncertainty of obtaining adequate protection by vegetative cover at many damsites, especially in arid regions, protection by cobbles or rock is preferred, and should be used where the cost is not prohibitive. Layers 24 inches thick are easier to place, but a 12-inch-thick layer usually affords sufficient protection.

If grasses are planted, those suitable for a given locality should be selected. Figure 127 shows the native grasses which have protected the downstream slope of Belle Fourche Dam of the Bureau of Reclamation from erosion for 40 years. Two drainage berms, one of which is shown in the photograph, are located on the downstream slope of this 115-foot-high dam. Usually fertilizer and uniform sprinkling of the seeded areas is necessary to promote the germination and foster the growth of grasses. Appendix G contains sample specifications for placing topsoil, planting seed, and watering the seeded area until completion of construction.

139. ***Surface Drainage.***—The desirability of providing facilities to take care of surface drainage on the abutments and valley floor is often overlooked in the design of earthfill dams. The result is that, although the upstream and downstream slopes and the crest of the dam are protected against erosion, unsightly gullying takes place at the contact of the embankment with earth abutments from which vegetation has been removed during the construction operations, especially if the abutments are steep.

This condition is most likely to develop along the contact of the downstream slope with the abutments. Gullying can usually be controlled by constructing a gutter along the contact. The gutter may be formed of cobbles or rock used in the downstream surfacing. If the downstream slope is seeded, a concrete, asphalt, or dry-rock paved gutter should be provided. The likelihood of gullying of the abutments and gentle slopes of the valley floor by runoff from the downstream slope of the dam also should be considered; contour ditches or open drains may be needed to control erosion. Figure 128 shows typical sections of a contour ditch and an open drain.

Attention should also be given to the construction of outfall drains or channels to conduct the toe drain discharge away from the downstream toe of the embankment so that an unsightly boggy area will not be created. The need for surface drainage facilities and the most appropriate type for a particular site can usually best be determined

Figure 127. **Downstream slope of Belle Fourche Dam protected by grass.**

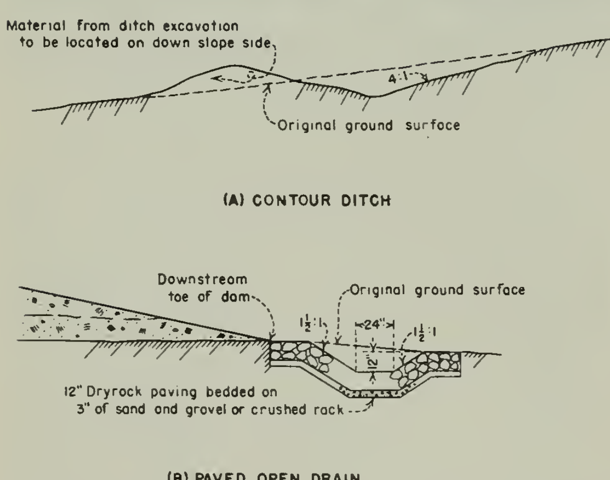


Figure 128. **Typical sections of contour ditch and open drain.**

by field examination prior to or during construction.

140. ***Flared Slopes at Abutments.***—If necessary, embankment slopes may be flared at the abutments to provide flatter slopes for stability or to control seepage through a longer contact of the impervious zone of the dam with the abutment. If the abutment is pervious and if a positive cutoff cannot be attained economically, it may be possible to obtain the effect of an upstream blanket by flaring the embankment. The design of the transition from normal to flared slopes is governed largely by the topography of the site, the length of contact desired, and the desirability of making a gradual transition without abrupt changes for ease of construction and for appearance.

F. DESIGN EXAMPLES OF SMALL EARTHFILL DAMS

141. ***General.***—The designs of 25 Bureau of Reclamation earthfill dams are discussed briefly in the next section. With only a few exceptions these dams are less than 50 feet in height above the original streambed, or are dikes of that size constructed in conjunction with larger dams. The few exceptions were included to illustrate designs for unusual conditions which were not encoun-

tered in the construction of any of the smaller dams. These exceptions, however, were chosen so that only dams less than 100 feet in height above the original streambed are involved.

These designs include small earthfill dams constructed by the Bureau of Reclamation since 1930. Only 4 of these dams were constructed prior to 1940, 8 were constructed in the 1940's, and the remaining 13 dams were completed between 1950 and 1958 or were under construction at the latter time. Many minor dikes constructed in conjunction with storage dams or canal systems were omitted because of their similarity to other designs which are shown.

The purpose of these examples is to illustrate the variety of designs that were conceived to meet widely varying conditions in foundations and in availability of construction materials. Without exception, the completed structures have given satisfactory service and no serious or costly maintenance problems have arisen. On the other hand, even the older designs do not appear to be unduly conservative and costly. It is believed that a designer of small earthfill dams can glean valuable ideas from a study of these examples.

142. *Maximum Sections.* Figures 129 to 154, inclusive, show the maximum sections of small earthfill dams constructed by the Bureau of Reclamation. A brief explanation of each of the designs follows:

(a) *Altus Reservoir North dike (fig. 129).*—This dike was constructed as part of a masonry dam. The cutoff trench width is narrower than recommended for modern practice; otherwise the design closely conforms to the recommendations made in this text. Note the downstream slope protection, the crest surfacing, and the foundation toe drain.

(b) *Anita Dam (fig. 130).*—This design conforms to modern practice except for the narrow bottom width of the cutoff trench and the narrow berms on the upstream and downstream slopes. Modern practice requires wider widths to accommodate equipment; furthermore, downstream slope berms are no longer common on low dams. Note the gravel downstream toe which effectively

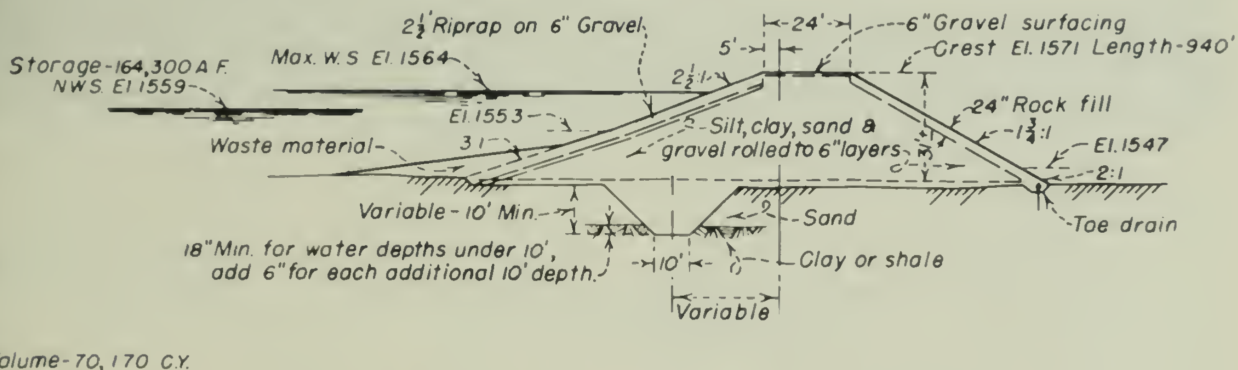


Figure 129. Altus Reservoir, North dike, located on the North Fork of the Red River, Okla. (Constructed 1940–42.) From drawing No. 103–D–581.

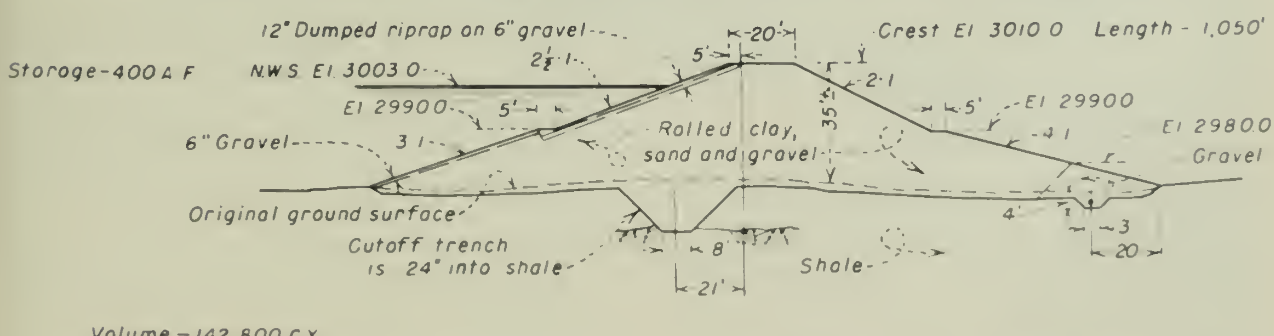


Figure 130. Anita Dam, located offstream from the Yellowstone River, Mont. (Constructed 1936.) From drawing No. 103–D–580.

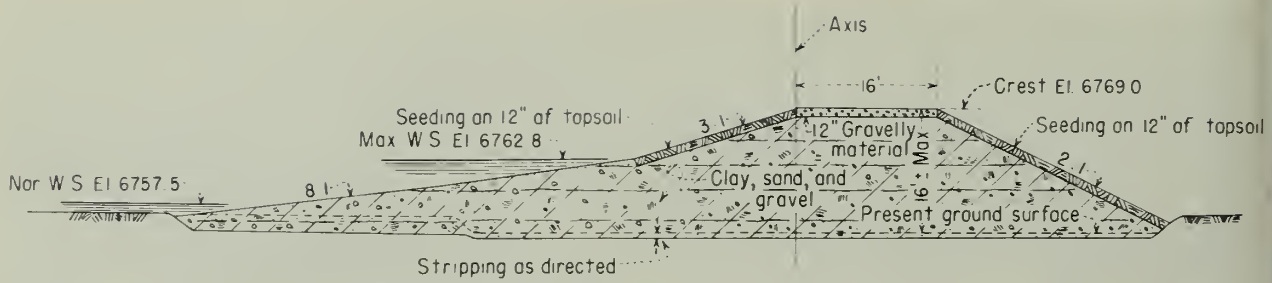


Figure 131. **Typical dike section, Big Sandy Dam, located on Big Sandy Creek, Wyo. (Constructed 1950–52.)**

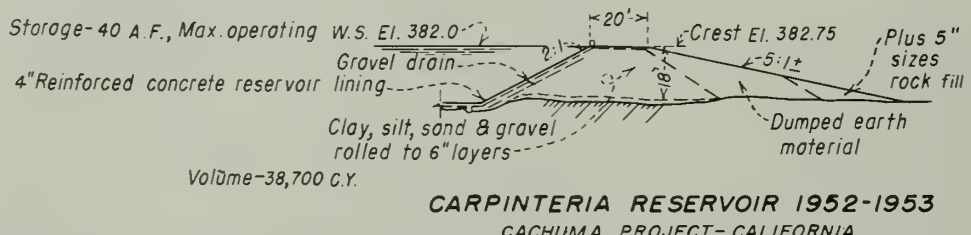


Figure 132. **Carpinteria Reservoir, terminal reservoir of a distribution system located near Carpinteria, Calif. (Constructed 1952–53.) From drawing No. 103–D–585.**

lowers the phreatic line in the embankment (see fig. 96(A)).

(c) *Big Sandy dike* (*fig. 131*).—This dike was constructed in conjunction with Big Sandy Dam (not shown), a 72-foot-high dam of conventional design with a 3 : 1 upstream slope protected by a 3-foot-thick layer of rock riprap. The design of the upstream slope of the dike represents a departure from usual design and was adopted because of the scarcity and expense of rock for riprap. Note that in the surcharge range the upstream slope of the dike is 8 : 1, which is the beaching slope of the embankment material. Freeboard above the maximum water surface is provided by a 3 : 1 slope which is planted to make it erosion resistant to wave splash and spray to which it will be subjected only rarely. This design is suitable for upstream slope protection of a detention dam, provided the maximum water surface will not be attained more than several times during the expected life of the dam.

(d) *Carpinteria Reservoir dike* (*fig. 132*).—Carpinteria Reservoir is a small equalizing reservoir constructed on a gently sloping sidehill by excavating on the uphill side and constructing a dike on the downhill side. A section of this dike is shown in the figure. The concrete lining is provided to prevent seepage which would be serious because of the location of the reservoir with respect to improved property. The concrete lining covers all the side slopes and the bottom of the reservoir. Figure 133 shows the reservoir lining being constructed. Because this reservoir is subject to rapid drawdown, a gravel drain is placed under the side lining to prevent uplift. A pipe drainage system is also provided under the reservoir floor lining.

The embankment was constructed of material from the excavation. This soil contained considerable rock fragments larger than 5 inches in diameter, and separation was required by the specifications in order to obtain an impervious zone which could be compacted satisfactorily. The dumped earth material zone provided for waste disposal of excess excavation. Oversize from the screening operation was used to construct the downstream rockfill toe.

(e) *Carter Lake Dam No. 3* (*fig. 134*).—This illustrates the design of a zoned embankment consisting of an earth impervious core and rock shells. At this site there was a limited amount of material for an impervious core, no sand-gravel, but a large amount of rock which could be quarried. Quarrying operations were controlled so as to produce the desired amount and gradation of rock fragments. The rockfill consists of rock with a maximum size of 1 cubic yard and sufficient smaller rocks to fill the voids. The quarry fines zone, which acts as a filter between the rockfill and the impervious core, consists of rock fines not

Figure 133. Construction of concrete lining at Carpinteria Reservoir. SB-3262-R2.

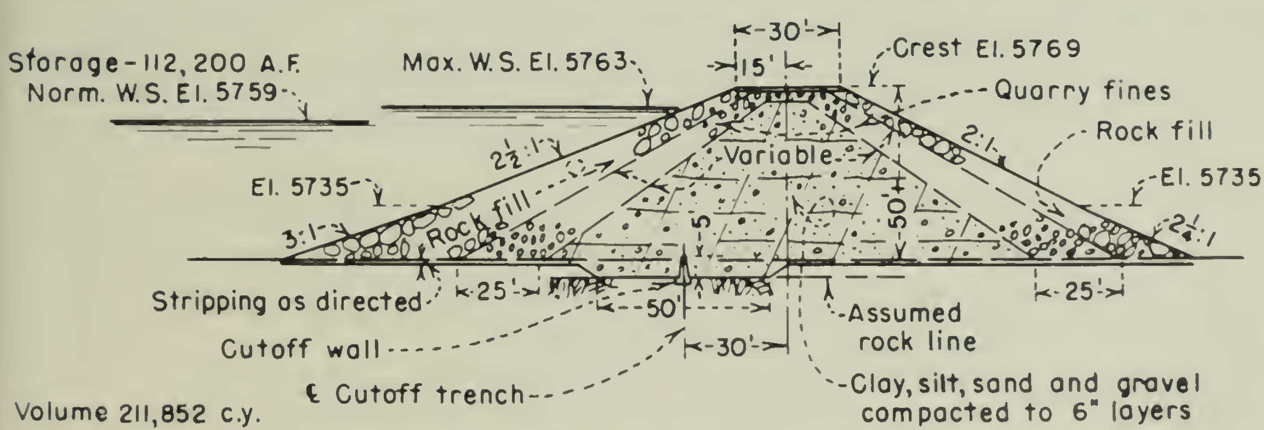


Figure 134. Carter Lake Dam No. 3, located on Dry Creek (a tributary of the Big Thompson River), Colo. (Constructed 1950–52.)

more than 20 percent of which pass a ¼-inch screen, with no pieces larger than 8 inches.

(f) *Crane Prairie Dam (fig. 135)*.—The design of this small dam is conventional. Except for bottom width of cutoff trench, the design conforms to the recommendations given in this text.

(g) *Crescent Lake Dam (fig. 136)*.—Crescent Lake Dam is a typical modern small zoned earthfill dam. The large pervious shells allow the use of steep slopes on the embankment. Note the key trench and the modification to the zone lines near the crest of the dam to facilitate construction.

484556 O -60—— 16

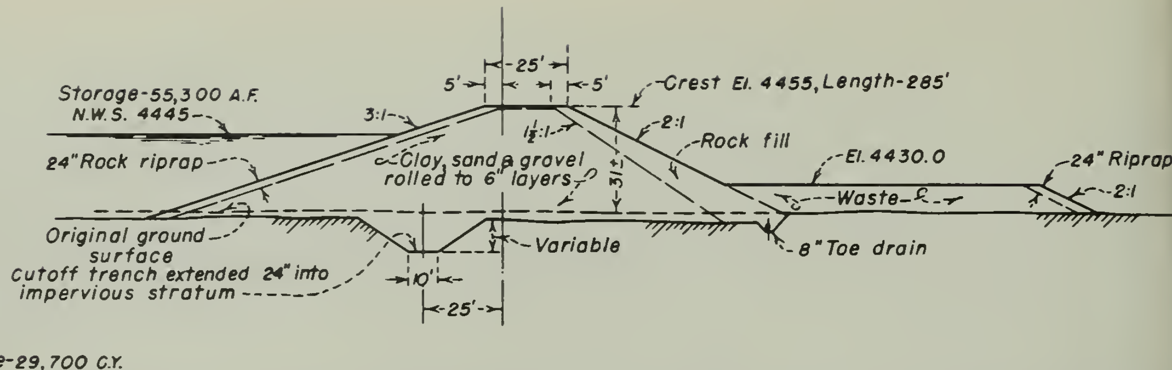


Figure 135. Crane Prairie Dam, located on the Deschutes River, Oreg. (Constructed 1939–40.) From drawing No. 103–D–581.

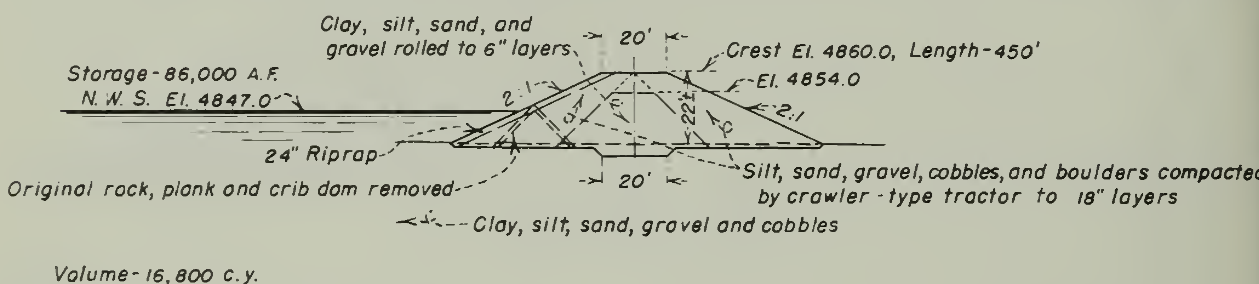


Figure 136. Crescent Lake Dam, located on Crescent Creek, Oreg. (Constructed 1954–56.) From drawing No. 103–D–586.

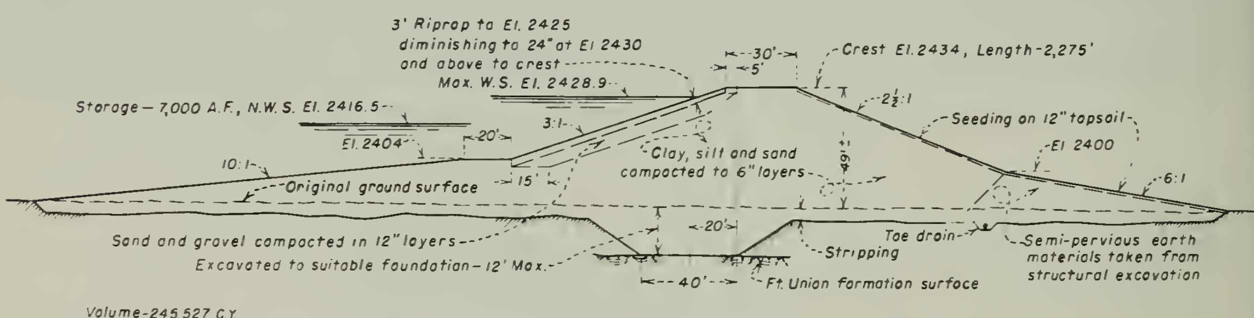


Figure 137. Dickinson Dam, located on the Heart River, N. Dak. (Constructed 1948–50.) From drawing No. 103–D–584.

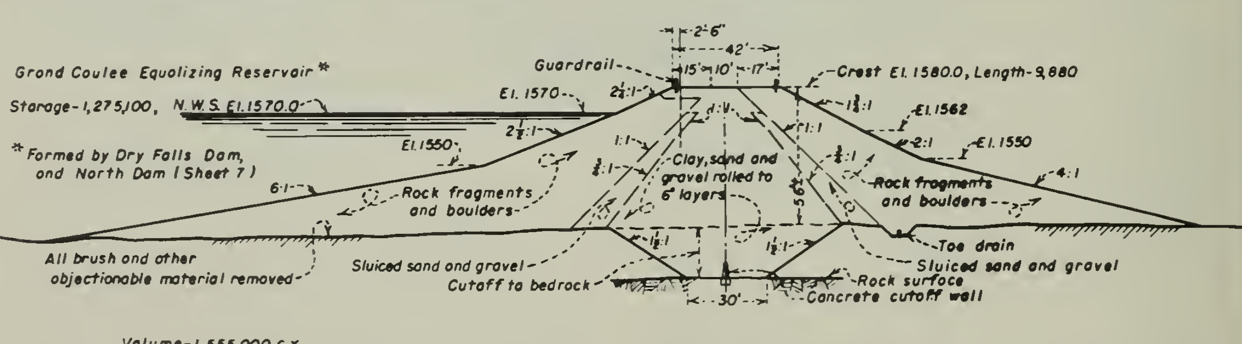


Figure 138. Dry Falls Dam, located near Coulee City, Wash. It forms the south barrier of Grand Coulee Equalizing Reservoir. (Constructed 1946–49.) From drawing No. 103–D–583.

(h) *Dickinson Dam* (*fig. 137*).—This dam is the modified homogeneous type. The flat slopes at the toes of the dam form stabilizing fills which were provided because of the unconsolidated and uncemented foundation material. Note the decrease in thickness of riprap near the crest of the dam. This was done to decrease the amount of costly rock, and in view of the infrequent exposure to wave action because of the large surcharge head.

(i) *Dry Falls Dam* (*fig. 138*).—This design illustrates a zoned embankment constructed on a soft foundation. This dam is unusual in that rock was used to construct the stabilizing fills formed by flattening the slopes of the dam. Usually, rock is too expensive to be used for this purpose, but in this instance it was excavated for a canal which heads at the dam. The stability afforded to the section by the heavy rock zones permits steep slopes for the upper part of the dam. Note the filter zone provided between the core and the rockfills and the modifications made to the zoning lines near the crest of the dam to facilitate construction and preserve a sufficiently long path of percolation through impervious material. The 42-foot-wide crest was required because this dam is used for a major highway crossing.

(j) *Fruitgrowers Dam* (*fig. 139*).—This is another example of a small earthfill dam whose design conforms to modern practices except for the narrow bottom width of the cutoff trench. The construction reports note that the bottom width, in general, was made 12 to 14 feet to accommodate equipment. The cutoff trench was extended to shale.

(k) *Howard Prairie Dam* (*fig. 140*).—Although this dam is higher than those within the scope of this text, it is included herein as an example of a zoned embankment with a relatively thin impervious core and with heavy rockfill supporting zones. The overburden penetrated by the cutoff trench consisted of topsoil and sand-gravel. Note the transition zones between the impervious core and the rockfill zones, and the modifications made to the zone lines near the crest of the dam to facilitate construction.

(l) *Lion Lake dikes* (*fig. 141*).—This is illustrative of a very small embankment constructed to impound a water supply reservoir. The trench shown is a relatively deep key extending into glacial deposits of considerable depth.

(m) *Lovewell Dam* (*fig. 142*).—Although this dam is somewhat higher than those within the scope of this text, it is included herein to illustrate the use of stabilizing fills on an extremely soft clay foundation. Note also how a minimum amount of riprap is utilized in this design. The 20 : 1 slope of the upstream stabilizing fill does not require protection. Only a minor amount of erosion is expected on the upstream $2\frac{1}{2}$: 1 slope of the stabilizing fill because the extremely short reservoir fetch below elevation 1575 will produce little wave action. Minor erosion of this extensive stabilizing fill will not be of consequence.

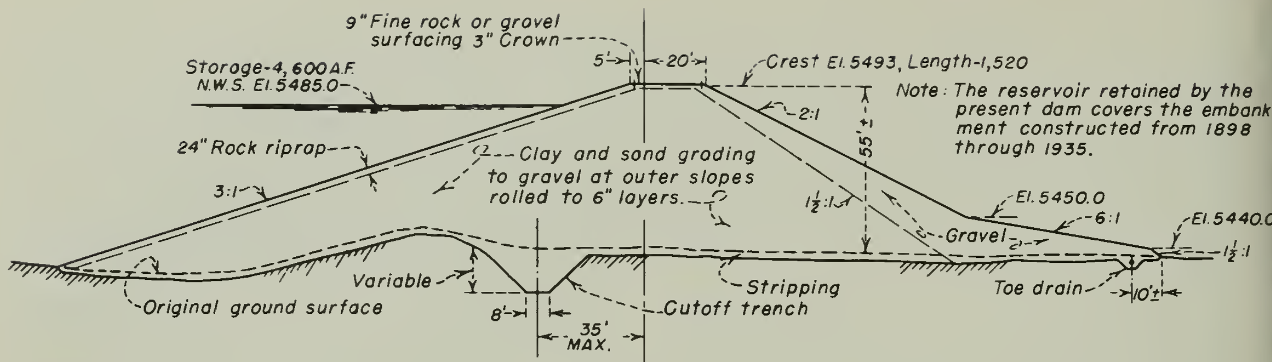
(n) *Marys Lake dikes* (*fig. 143*).—This design does not provide a pervious section at the downstream toe to lower the phreatic line in the embankment. It was not considered necessary because of the low level of the maximum operating water surface. The cutoff trench is narrower than recommended for modern practice, and the cutoff wall probably would be omitted if a new design were made for this site.

(o) *Midview Dam* (*fig. 144*).—This design conforms to modern practice except for the cutoff trench, which is considered to be too narrow for economical utilization of equipment, and for the cutoff wall which likely would be replaced by a grout cap.

(p) *Olympus Dam* (*fig. 145*).—This design is an example of an earthfill dam with multiple zones. Note the upstream slope protection; the riprap is used only on the upper portion of the slope where heavy wave action is to be expected. The lower portion of the slope is protected by rock and cobble fill because the water level will rarely be lowered into this range.

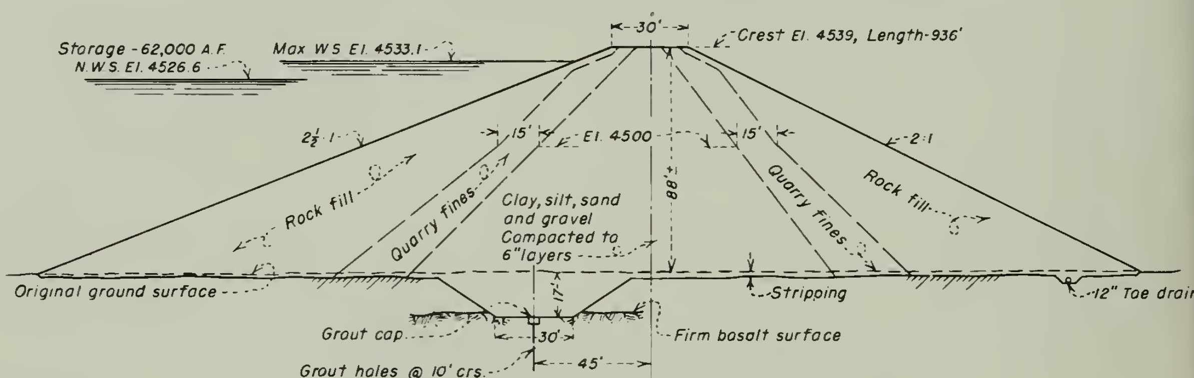
(q) *Picacho North Dam* (*fig. 146*).—This is a detention dam which has no permanent storage pool. It is constructed on a stratified pervious foundation. The impervious zone of the dam was extended to the upstream toe and was made continuous with waste placed upstream from the dam in order to increase the path of percolation through the foundation. To facilitate use of available materials from stratified deposits, the design of the embankment was based on the impervious core varying between the slope limits shown.

(r) *Picacho South Dam* (*fig. 147*).—This dam is also a detention type dam with no permanent storage pool. At this site the foundation was



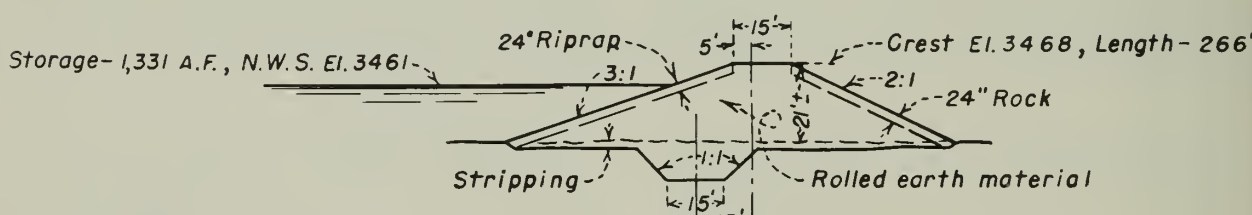
Volume-135,500 C.Y.

Figure 139. **Fruitgrowers Dam, located on Alfalfa Run Wash in Colorado. Its main water supply is derived from Currant and Surface Creeks. (Constructed 1938–39.) From drawing No. 103–D–581.**



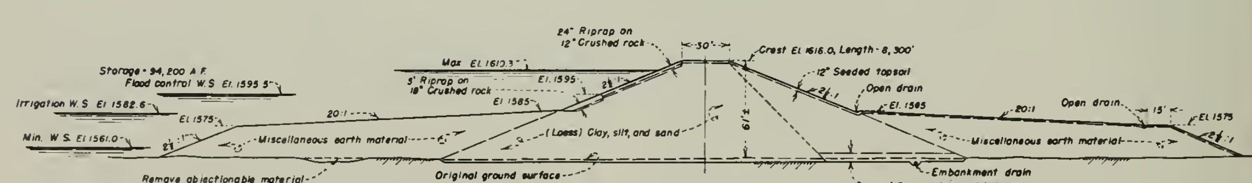
Volume - 406,000 C.Y.

Figure 140. **Howard Prairie Dam, located on Beaver Creek (a tributary of Jenny Creek), Oreg. (Constructed 1957–59.) From drawing No. 103–D–587.**



Volume- 11,000 C.Y.

Figure 141. **Lion Lake dikes, constructed for a water supply reservoir for Hungry Horse Dam Government camp, Mont. (Constructed 1947.) From drawing No. 103–D–583.**



Volume - 5,174,700 c.y.

Figure 142. **Lovewell Dam, located on the Republican River (offstream from White Rock Creek), Kans. (Constructed 1954–57.) From drawing No. 103–D–586.**

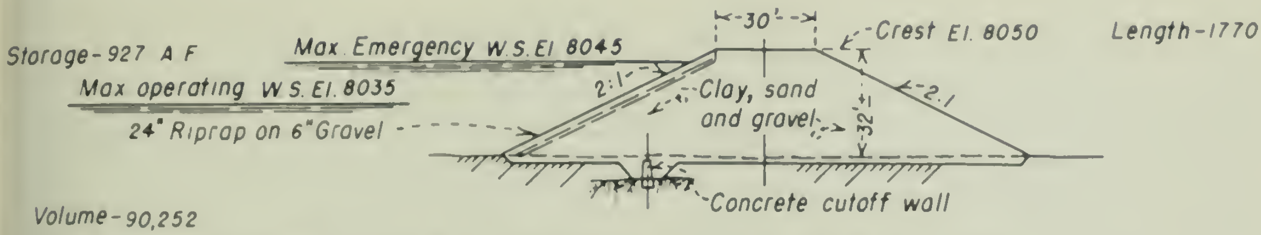


Figure 143. Marys Lake dikes, located offstream from Fish Creek (a tributary of the Big Thompson River), Colo. (Constructed 1947–48.) From drawing No. 103–D–582.

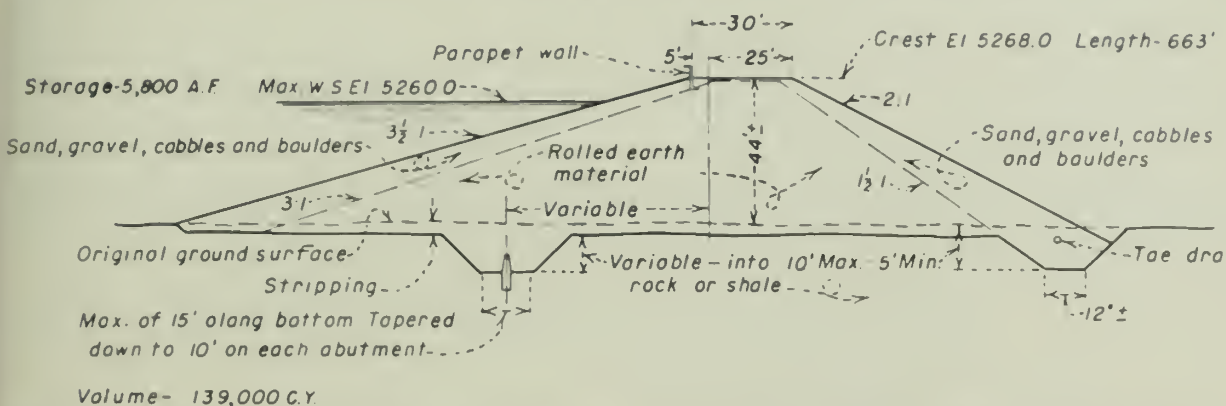


Figure 144. Midview Dam, located offstream from the Duchesne River, Utah. (Constructed 1935–37.) From drawing No. 103–D–580.

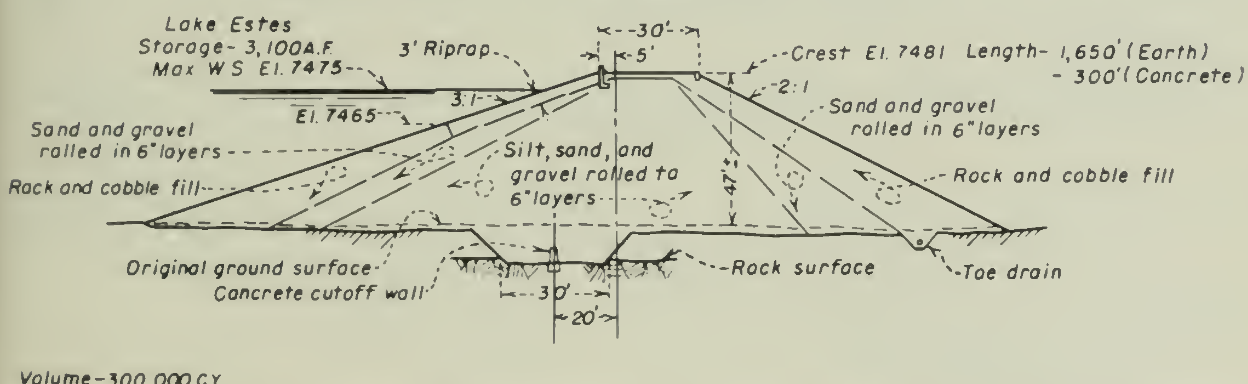


Figure 145. Olympus Dam, located on the Big Thompson River, Colo. (Constructed 1947–49.) From drawing No. 103–D–583.

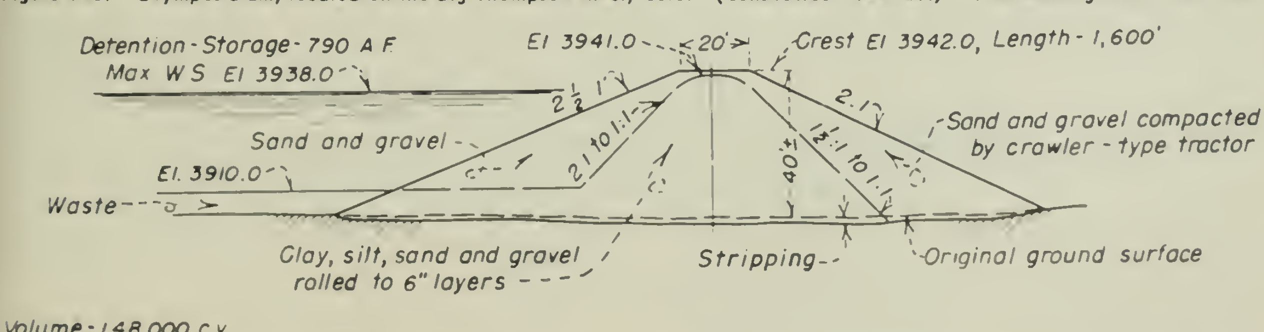


Figure 146. Picacho North Dam, a detention dam located on the North Branch of Picacho Arroyo, N.Mex. (Constructed 1953–54.) From drawing No. 103–D–586.

relatively impervious; only sufficient pervious material for slope protection of the embankment could be found in the vicinity. A 10-foot-wide sand-gravel-cobble zone was specified on both the upstream and downstream slopes to facilitate compaction. Riprap was not considered necessary as the outlet capacity is sufficient to evacuate the reservoir within a period of a few days time.

(s) *Pishkun dikes* (*fig. 148*).—This is an interesting design in that it illustrates how a 43-foot-high dam was raised 6 feet ten years after completion of the initial construction. This embankment is essentially homogeneous; a small downstream pervious zone was provided in the original design to contain the more pervious materials found in the borrow pit. Note the sparing use of costly riprap; it is provided only in the operating range of the reservoir where wave action will be most severe.

(t) *San Jacinto Regulating Reservoir* (*fig. 149*).—This dam is the diaphragm type. A scarcity of impervious material made this design necessary. It consists of relatively pervious sand secured from reservoir excavation with a thin upstream impervious soil blanket protected by riprap with an interposed filter. This type of design is not recommended generally for small dams (see sec. 120).

(u) *Shadow Mountain Dam* (*fig. 150*).—This dam has a pervious glacial foundation. The design provides a partial cutoff trench and a flat upstream slope which functions as a blanket to reduce seepage, and a drainage blanket in the downstream portion of the dam to control seepage uplift.

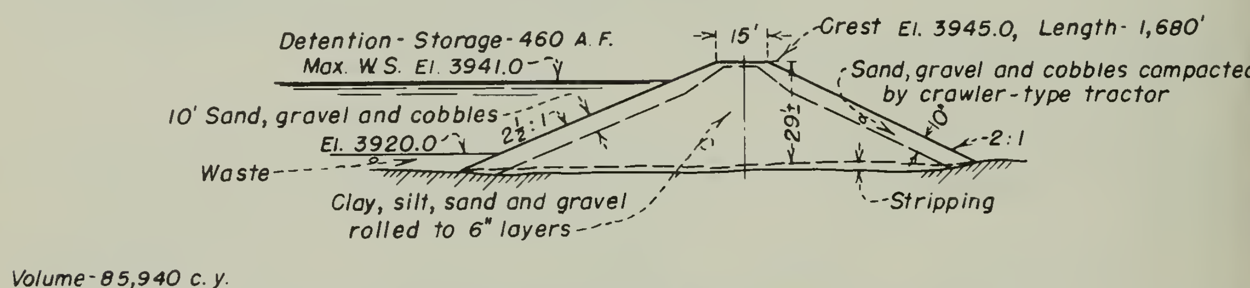


Figure 147. Picacho South Dam, a detention dam located on the South Branch of Picacho Arroyo, N.Mex. (Constructed 1953–54.) From drawing No. 103–D–586.

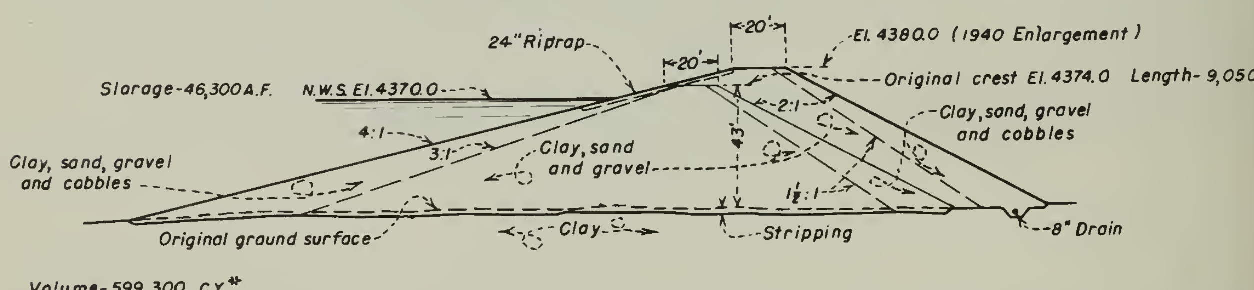


Figure 148. Pishkun dikes, located offstream from the Sun River, Mont. (Constructed 1930–31.) From drawing No. 103–D–580

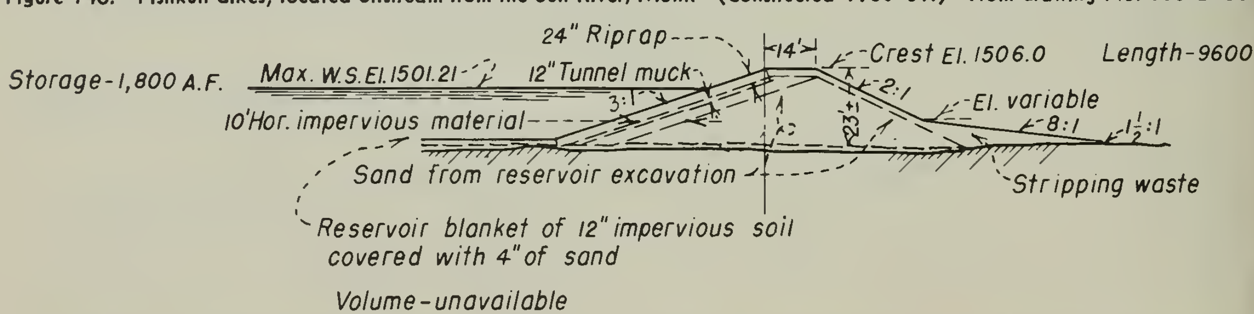


Figure 149. San Jacinto Regulating Reservoir, located on the San Diego Aqueduct, Calif. (Constructed 1945.) From drawing No. 103–D–582.

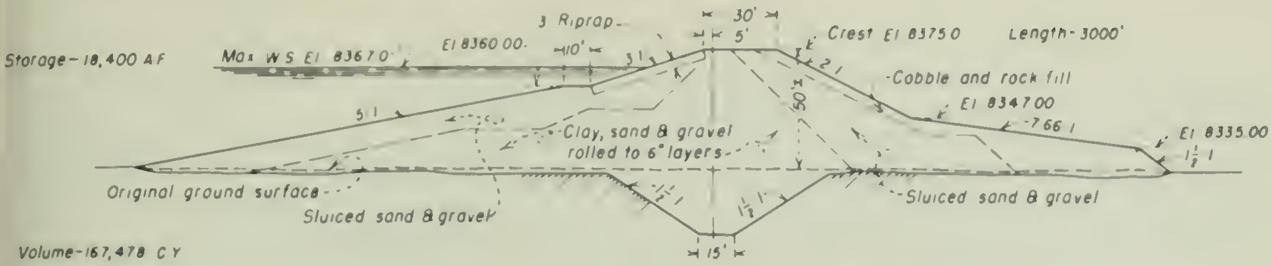


Figure 150. Shadow Mountain Dam, located on the Colorado River, Colo. (Constructed 1943–46.) From drawing No. 103–D–582.

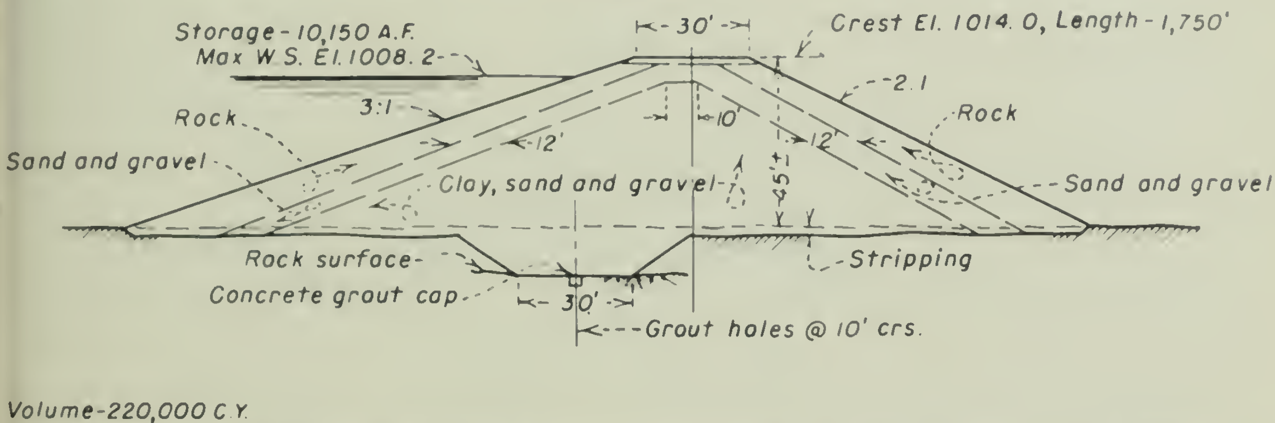


Figure 151. Soda Lake dike, located offstream from the Columbia River, Wash. (Constructed 1950–52.) From drawing No. 103–D–584.

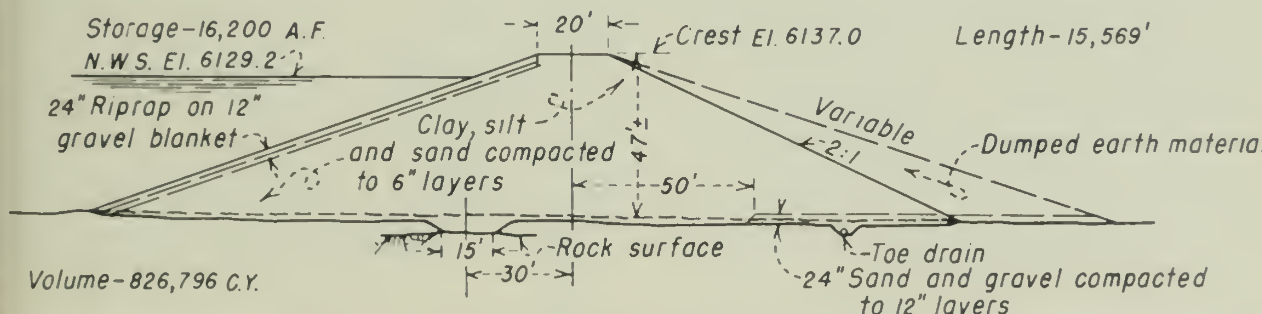


Figure 152. Stubblefield Dam, located offstream from the Vermejo River, N.Mex. (Constructed 1953–54.) From drawing No. 103–D–585.

(v) *Soda Lake dike* (*fig. 151*).—In all respects this is a well-designed small dam by the standards given in this text. The filter zones between the impervious core and the outer rockfill zones were made 12 feet wide, and the zone lines were modified near the crest of the dam to facilitate construction.

(w) *Stubblefield Dam* (*fig. 152*).—The design is typical of a homogeneous dam modified by a horizontal filter drain to lower the phreatic line in the downstream portion of the embankment. The dumped earth shown outside of the 2 : 1 downstream slope was for disposal of waste material. It was considered that disposal of waste material (including organic material) in this manner not only would flatten the downstream slope but would also develop a vegetative cover which would be adequate for slope protection.

(x) *Tiber dike* (*fig. 153*).—The upper view of the illustration shows the maximum section of Tiber dike, which is conventional in design. A key trench 10 feet deep was excavated into the glacial

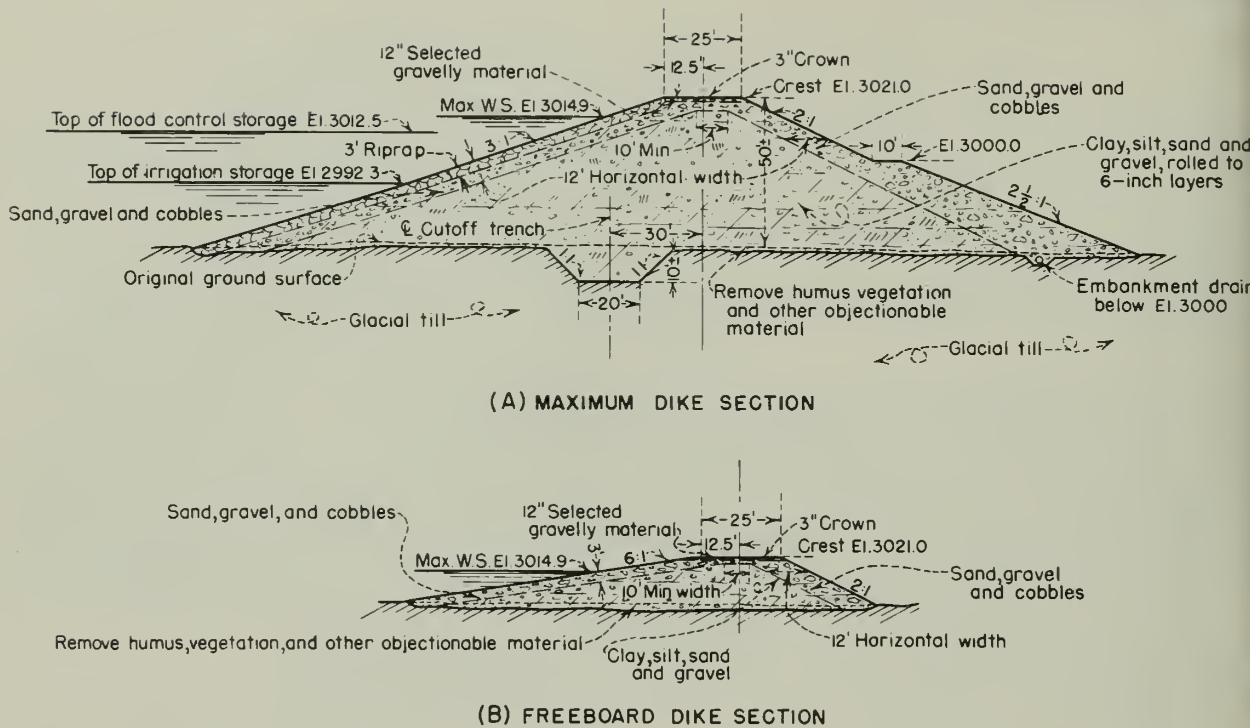


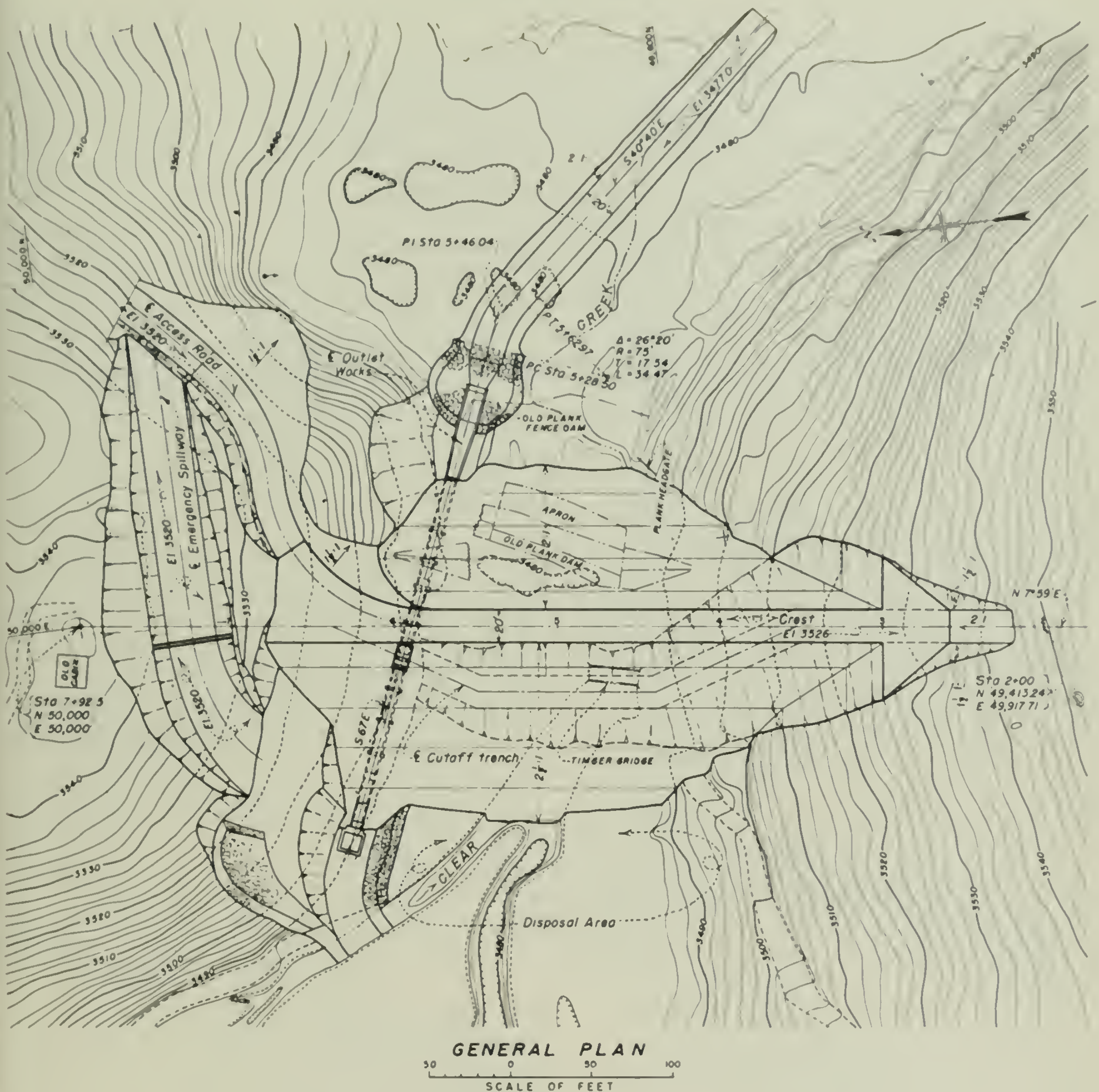
Figure 153. **Dike sections of Tiber Dam, located on the Marias River, Mont. (Constructed 1953–56.)**

till foundation, and the upstream slope is protected by riprap. The lower view shows modifications made to the dike where the original ground is above elevation 2995, which is above the top of the irrigation storage at elevation 2992.3. Because the slope will be subjected to wave action only at time of flood, it was decided that adequate protection would be provided by a 3-foot-thick layer of compacted sand, gravel, and cobbles (maximum size 10 inches) if the slope were flattened to 6 : 1. This modification was economical because of the high cost for riprap. It illustrates design flexibility in achieving the most economical structure.

(y) *Wasco Dam (fig. 154)*.—Wasco Dam (under construction 1958) is used as an example throughout this text.[4] It is not unusual in design and it may be considered typical of small zoned earthfill dams constructed on pervious shallow foundations which can economically be cut off by open trench methods. The cobble and rockfill zones utilize oversize rock removed from impervious soil and rock fragments from excavations for appurtenant structures. A toe drain was not used in the design because the foundation overburden is definitely pervious.

---

[4] The materials distribution chart for Wasco Dam is shown on fig. 119.



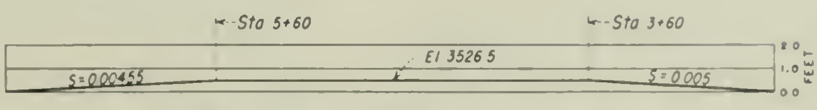
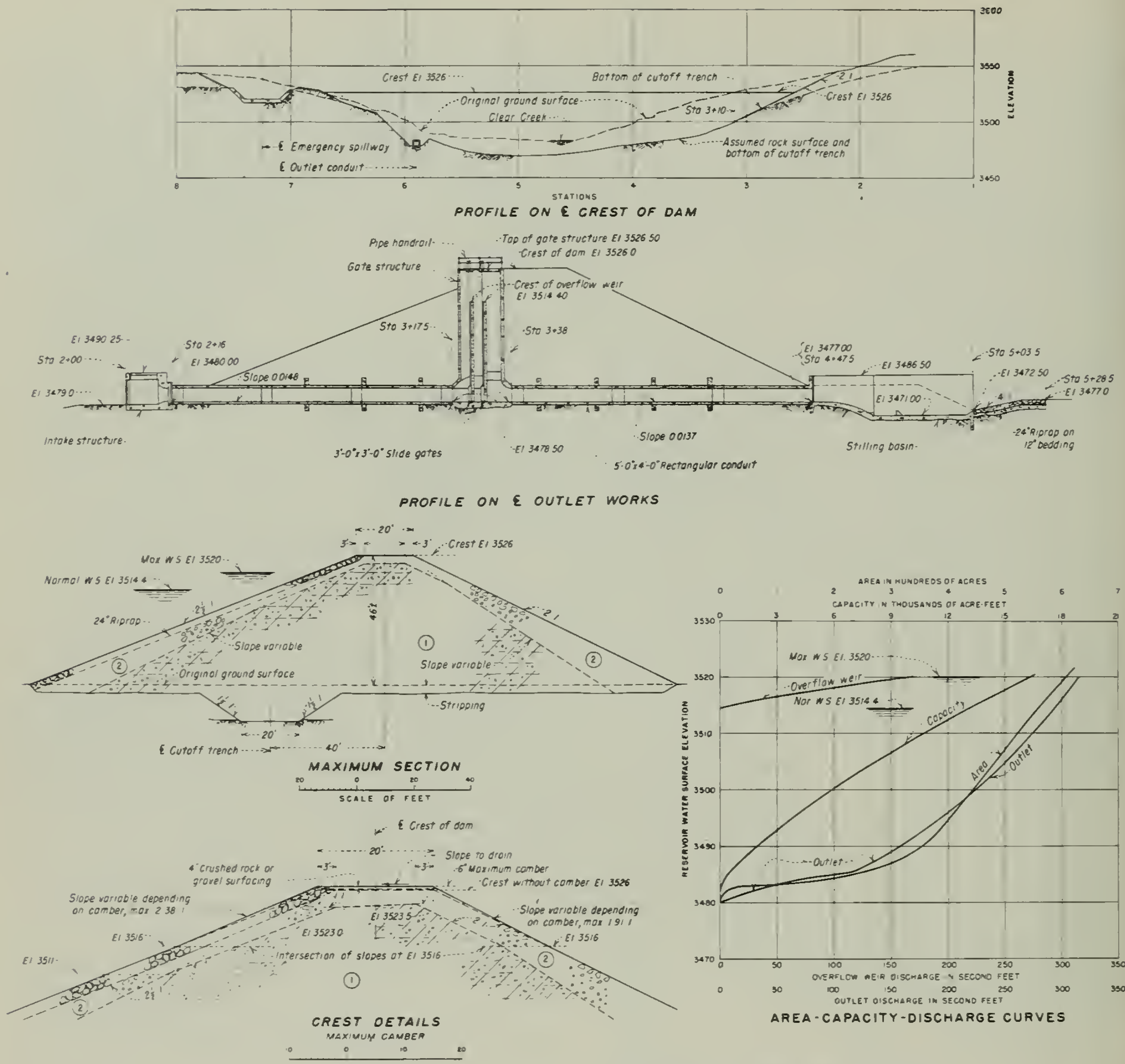


DIAGRAM FOR CAMBER ON CREST OF DAM

Figure 154. General plan and sections, Wasco Dam, located on Clear Creek (a tributary of White River), Oreg. From drawing No. 350–D–4. (Sheet 1 of 2.)



RESERVOIR STORAGE ALLOCATION

PURPOSE	ELEVATION	STORAGE ACRE-FEET
Conservation	3488 to 3514 4	11,860
Inactive	Stream bed to El 3488	1,200
Total storage capacity		13,060

Surcharge of 3050 A F (Max W S El 3520) in combination with outlet capacity of 320 c f s is provided to protect against the inflow design flood having a peak of 1,150 c f s and 15 day volume of 12,000 A F

EMBANKMENT EXPLANATION

(1) Selected clay, silt, and sand compacted by tamping rollers to 6-inch layers
(2) Cobble and rockfill.

NOTE

All existing structures interfering with required construction are to be removed

Figure 154. General plan and sections, Wasco Dam, located on Clear Creek (a tributary of White River), Oreg. From drawing No. 350-D-4. (Sheet 2 of 2.)

G. BIBLIOGRAPHY

143. *Bibliography.*

[1] Schuyler, J. D., "Reservoirs for Irrigation," John Wiley & Sons, Inc., New York, N.Y., 1905.

[2] Justin, J. D., "Earth Dams Projects," John Wiley & Sons, Inc., New York, N.Y., 1932.

[3] Bassell, Burr, "Earth Dams," Engineering News Publishing Co., New York, N.Y., 1907.

[4] Justin, J. D., Hinds, Julian, and Creager, W. P., "Engineering for Dams," John Wiley & Sons, Inc., New York, N.Y., vol. III, p. 675.

[5] Task Committees on Cement and Clay Grouting, "Symposium on Cement and Clay Grouting of Foundations," ASCE Proceedings, vol. 84, February 1958, Journal of the Soil Mechanics and Foundations Division, No. SM1, papers No. 1544 to 1552.

[6] Gibbs, H. J., and Holtz, W. G., "Research on Determining the Density of Sands by Spoon Penetration Testing," Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering, London, 1957, vol. I, p. 38 (adapted from fig. 7).

[7] Terzaghi, Karl, and Peck, R. B., "Soil Mechanics in Engineering Practice," John Wiley & Sons, Inc., New York, N.Y., 1948, p. 230.

[8] Rice, O. L., and Arthur, H. G., "The Most Recent Methods Developed to Avoid Piping or Blow-Outs in Dams," Third Congress on Large Dams, Stockholm, Sweden, 1948, vol. II, Question No. 10, R. 49.

[9] Casagrande, Arthur, "Seepage Through Dams," Journal New England Water Works Association, June 1937, p. 131.

[10] Taylor, D. W., "Fundamentals of Soil Mechanics," John Wiley & Sons, Inc., New York, N.Y., 1948, p. 156.

[11] Creager, W. P., Justin, J. D., and Hinds, Julian, "Engineering for Dams," John Wiley & Sons, Inc., New York, N.Y., vols. I and III, 1945.

[12] Justin, J. D., Hinds, Julian, and Creager, W. P., "Engineering for Dams," John Wiley & Sons, Inc., New York, N.Y., vol. III, p. 695.

[13] "Rotary Mixed-In-Place Grouting Found Feasible," Civil Engineering, vol. 23, February 1953, p. 126.

[14] Task Committee on Chemical Grouting, "Chemical Grouting—Progress Report," ASCE Proceedings, vol. 83, November 1957, Journal of the Soil Mechanics and Foundations Division, No. SM4, paper No. 1426.

[15] Bennett, P. T., "The Effect of Blankets on Seepage Through Pervious Foundations," Trans. ASCE, vol. III, 1946, p. 215.

[16] Terzaghi, Karl, and Peck, R. B., "Soil Mechanics in Engineering Practice," John Wiley & Sons, Inc., New York, N.Y., 1948, p. 228.

[17] Bertram, G. E., "An Experimental Investigation of Protective Filters," Graduate School of Engineering, Harvard University, Soil Mechanics Series No. 7, January 1940.

[18] "Soil Mechanics Fact Finding Survey, Seepage Studies; Progress Report," Technical Memorandum No. 175–1, U.S. Waterways Experiment Station, Vicksburg, Miss., March 1, 1941.

[19] "The Use of Laboratory Tests to Develop Design Criteria for Protective Filters," Earth Laboratory Report No. EM–425, Bureau of Reclamation, June 20, 1955.

[20] Middlebrooks, T. A., and Jervis, W. H., "Relief Wells for Dams and Levees," Trans. ASCE, vol. 112, 1947, p. 1321.

[21] Turnbull, W. J., and Mansur, C. I., "Relief Well Systems for Dams and Levees," Trans. ASCE, vol. 119, 1954, p. 842.

[22] Mansur, C. I., and Kaufman, R. I., "Control of Underseepage, Mississippi River Levees, St. Louis District," ASCE Proceedings, vol. 82, January 1956, Journal of the Soil Mechanics and Foundations Division, No. SM1, paper No. 864.

[23] Clevenger, W. A., "Experiences With Loess as a Foundation Material," ASCE Proceedings, vol. 82, July 1956, Journal of the Soil Mechanics and Foundations Division, No. SM3, paper No. 1025.

[24] Terzaghi, Karl, "Simple Tests Determine Hydrostatic Uplift," Engineering News-Record, June 18, 1936, p. 872.

[25] Timoshenko, S., and Goodier, J. N., "Theory of Elasticity," McGraw-Hill Book Co., Inc., New York, N.Y., 1951, p. 14.

[26] Hilf, J. W., "Estimating Construction Pore Pressures in Rolled Earth Dams," Proceedings of the Second Conference on Soil Mechanics and Foundations Engineering, Rotterdam, June 1948, vol. III, p. 234.

[27] May, D. R., "Application of the Planimeter to the Swedish Method of Analyzing the Stability of Earth Slopes," Transactions, Second Congress on Large Dams, Washington, D.C., pt. I of appx. A, vol. IV, 1936, p. 540.

[28] "Review of Slope Protection Methods," Subcommittee on Slope Protection, Soil Mechanics and Foundations Division, Proceedings ASCE, vol. 74, June 1948.

[29] "Slope Protection for Earth Dams," Preliminary Report, U.S. Corps of Engineers, Waterways Experiment Station, Vicksburg, Miss., March 1949.

Rockfill Dams

R. W. BOCK [1]

A. GENERAL

144. ***Origin and Usage.***—Rockfill dams are generally conceded to have had their origin about 100 years ago during the California Gold Rush. The most active period of construction of rockfill dams was from the late 1800's to the mid-1930's. A detailed description of the design and construction of a number of these dams is given in Galloway's paper entitled "The Design of Rockfill Dams" [1].[2] A number of major rockfill structures were constructed in the 1950's. The cost of producing large quantities of rock for the construction of rockfill dams makes this type of dam economical only in remote areas where the cost of concrete would be high or in areas where there is a scarcity of earthfill materials and the only material for construction of the structure consists of durable, hard rock.

145. ***Definition.***—A rockfill dam is an embankment which uses variable sizes of rock to provide stability and an impervious membrane to provide watertightness. Many different materials have been used for the membrane, including earth, concrete, steel, asphalt, and wood.

Although successful dams have been constructed with internal or "buried" diaphragms, this type of construction is not recommended for structures within the scope of this text. The construction of an internal diaphragm of earth with the necessary filters requires a higher degree of precision and closer control than it is feasible to obtain for small dams. Internal diaphragms of rigid material such as concrete have the disadvantage of not being readily available for inspection or emergency repair if they are ruptured through settlement of the dam or its foundation.

An earth blanket on the upstream slope of an otherwise pervious dam also is not recommended because of the expense and the difficulty of constructing suitable filters. Furthermore, the earth blanket must be protected from erosion by wave action, so it is buried and not readily available for inspection or repair.

The watertight membrane for a small rockfill dam should be constructed on the upstream slope where its condition can be inspected when the reservoir is drawn down, and repairs made as necessary. Usually, the membrane will consist of portland cement concrete, although steel plates or wood planking have been successfully used to the extent of the limited life of those materials. Recently (1957) asphaltic concrete pavement has been used but there is no service history for such construction in connection with a rockfill dam. Regardless of the type of membrane used, a rockfill dam is not recommended when the normal reservoir operation does not permit periodic inspection of the membrane and repair if required.

146. ***Foundation Requirements.***—The foundation requirements for a rockfill dam are less severe than for a concrete gravity dam, but more severe than for an earthfill dam. Rockfill dams require foundations which will result in a minimum of settlement. For foundations other than rock, a specialist should be consulted as to their adequacy. Rock foundations should consist of hard, durable rock which cannot be softened or eroded appreciably by water percolating from the reservoir.

[1] Engineer, Earth Dams Section, Bureau of Reclamation.

[2] Numbers in brackets refer to items in the bibliography, sec. 156.

B. FOUNDATION DESIGN

The foundation should be free from faults, shear zones, or other structural weaknesses. Silt, clay, sand, and organic material must be removed from the foundation area before construction of the rockfill.

147. ***Cutoff Wall.***—A watertight seal must be provided along the contact of the impervious membrane with the foundation and abutments at the upstream toe of the dam to prevent seepage under the dam. In existing dams, this seal has been in the form of a concrete cutoff wall which extends from the upstream toe of the dam into bedrock. Figures 155, 156, and 157 show details of the attachment of various types of impervious membranes to the cutoff wall.

The width of the cutoff wall is usually governed by construction considerations. The depth of penetration of the cutoff wall into bedrock depends upon the character of the foundation rock. If the rock is sound, the cutoff wall should extend into the foundation rock not less than 3 feet. A deeper wall or special treatment such as grouting may be required if the rock is not sound, or if open joints or broken rock structure exists. Grouting should be planned regardless of the apparent quality of the rock until sufficient drilling exploration has been performed to demonstrate that there are no seams, joints, faults, or fissures in the bedrock which may result in leakage under the cutoff wall. Conservativeness is very desirable in this respect because of the difficulty and expense of finding and repairing leakage after the dam goes into operation. Grouting procedures for rock foundations are discussed in chapter V.

In addition to its function of preventing underseepage, the cutoff wall must provide adequate support for the weight and thrust of the membrane.

C. EMBANKMENT DESIGN

148. ***Selection of Rock Materials.***—Of prime importance to the success of a rockfill dam is the type of rock which is used for the rockfill zone. For economy, the rock must be located near the dam site; it may be obtained from quarry operations or from talus deposits. The rock must be hard, durable stone which will resist excessive breakdown during the hauling and placing operation. The rock also should withstand disintegration under the action of freezing and thawing. It should be essentially free of unstable minerals that would weather mechanically or chemically, causing the rock to disintegrate.

Most unweathered igneous and metamorphic rocks are of satisfactory quality for rockfill. Although a few sedimentary rocks are satisfactory for use, as a general rule they should be avoided. Part D of chapter IV gives the classification and engineering properties of rocks. The rock produced in the quarry or obtained from natural sources should be well graded from ½ cubic foot to 1 cubic yard in size and should contain less fines than sufficient to fill the voids. Slabby rocks should not be used as they tend to bridge, causing large voids. As the load is increased by construction of the rockfill, the bridging rocks may break, resulting in excessive settlement.

149. ***Dam Section.***—Earlier rockfill dams were constructed with steep upstream and downstream slopes to minimize the volume of rockfill; slopes as steep as ¾ to 1 were used. Since these slopes are considerably steeper than the natural slope of dumped rock, they were stabilized by thick zones of dry rubble masonry. Later designs eliminated the rubble masonry on the downstream slope by flattening the slope to the angle of repose of the rock, but the very steep upstream slope was retained. Other designs evolved in which the upstream slope was flattened somewhat so that the rubble masonry zone could be reduced, the downstream slope being allowed to take the natural angle of repose of dumped rock.

Inasmuch as stability from sliding is not a design consideration in a small rockfill dam because of its mass and weight, the determination of the external slopes depends upon the relative cost of dumped rockfill and rubble masonry. Slopes steeper than the natural slope of dumped rock are not economical under modern construction conditions. However, there is no advantage

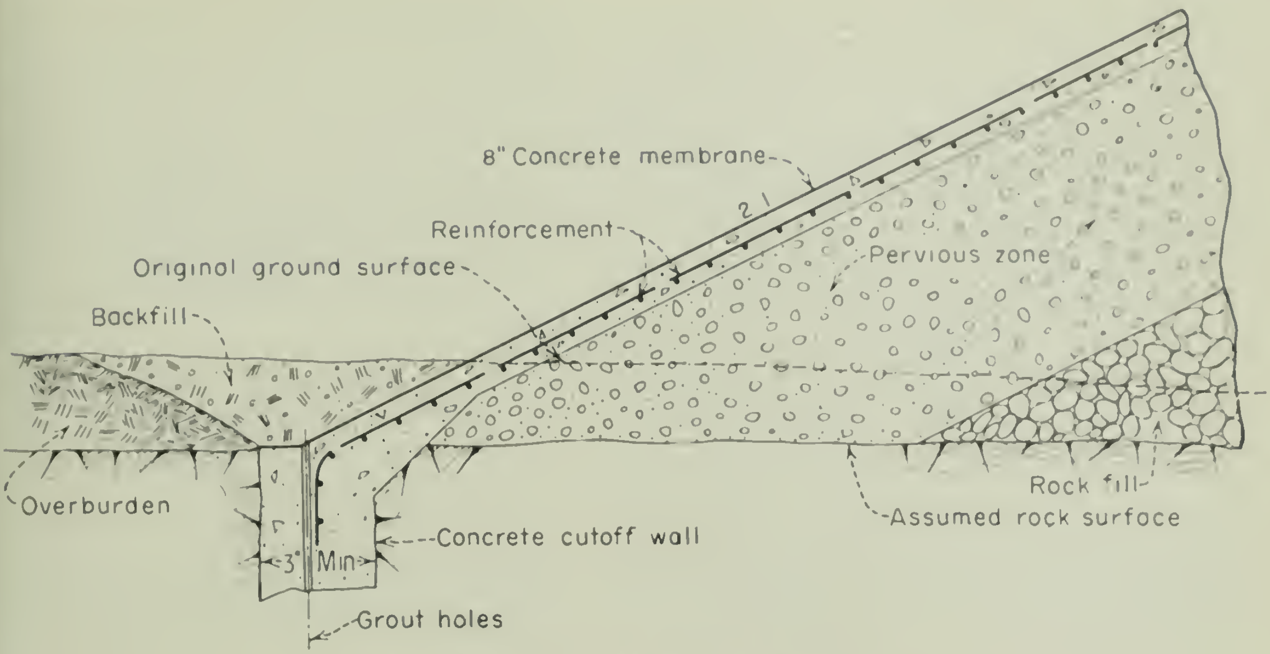


Figure 155. **Detail of concrete membrane at cutoff wall.**

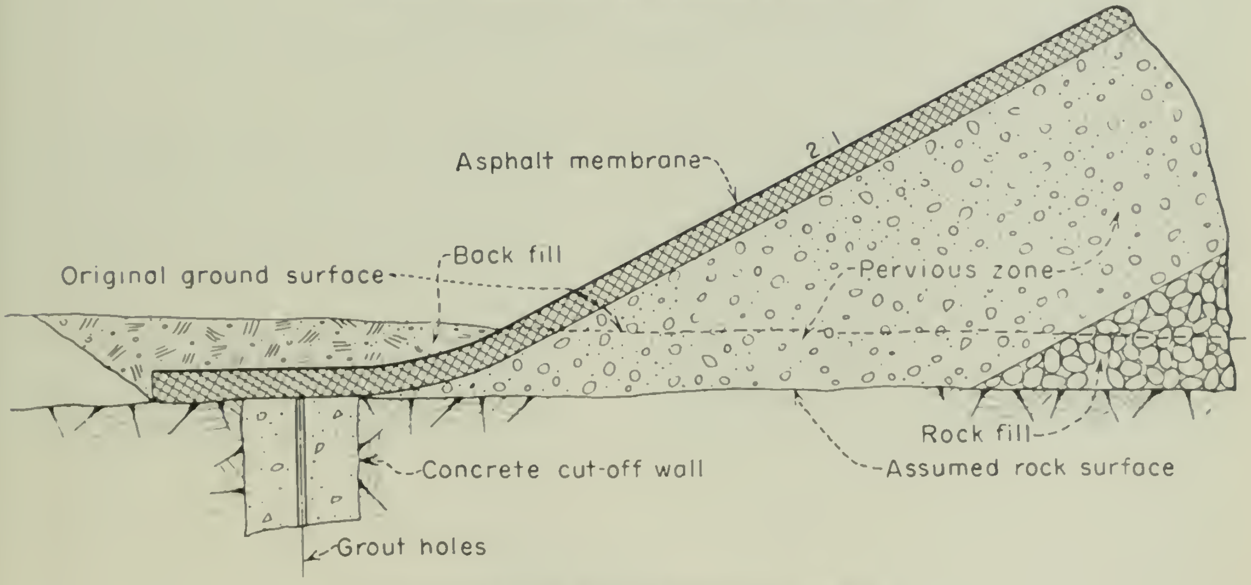


Figure 156. **Detail of asphalt membrane at cutoff wall.**

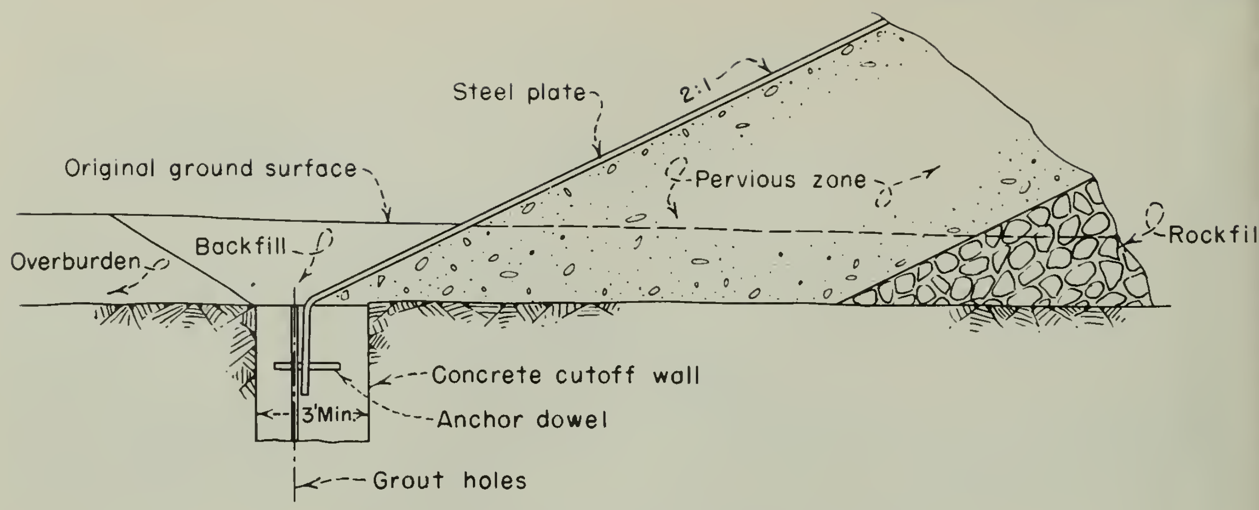


Figure 157. **Detail of steel plate membrane at cutoff wall.**

to slopes flatter than the natural slope except for facilitating construction of the upstream impervious facing.

For small rockfill dams, the downstream slope should be equal to the angle of repose of the dumped rockfill (about 1.4 to 1) and the upstream slope should be 2 to 1 to facilitate construction of the upstream impervious facing. The upstream face of the dam should be constructed slightly convex so that settlement in the fill will tend to close rather than open contraction joints in the impervious membrane. A typical section for a small rockfill dam is shown in figure 158.

Crest width, crest camber, freeboard requirements and other crest details for a rockfill dam are governed by the same considerations as those for an earthfill dam, which are given in chapter V.

150. *Rockfill Zone.*—The placement of the rockfill zone is one of the most important operations in the construction of a rockfill dam, as it is essential to minimize total settlement and the possibility of damage to the impervious membrane. Settlement of rockfills takes place in two stages. The first major settlement occurs during the construction of the rockfill. This stage of settlement has a minor bearing on the security of the impervious membrane, provided the membrane is not placed concurrently with the rockfill; on small dams the membrane should be placed after completion of the rockfill zone when the major settlement due to weight of the rockfill has taken place. The second major stage of settlement occurs as the reservoir fills and the thrust due to waterload is transmitted to the rockfill.

Rockfill in many of the existing large rockfill dams was placed by dumping in lifts varying from 75 to 150 feet high. In addition, water from high-velocity nozzles was used to sluice the rock-

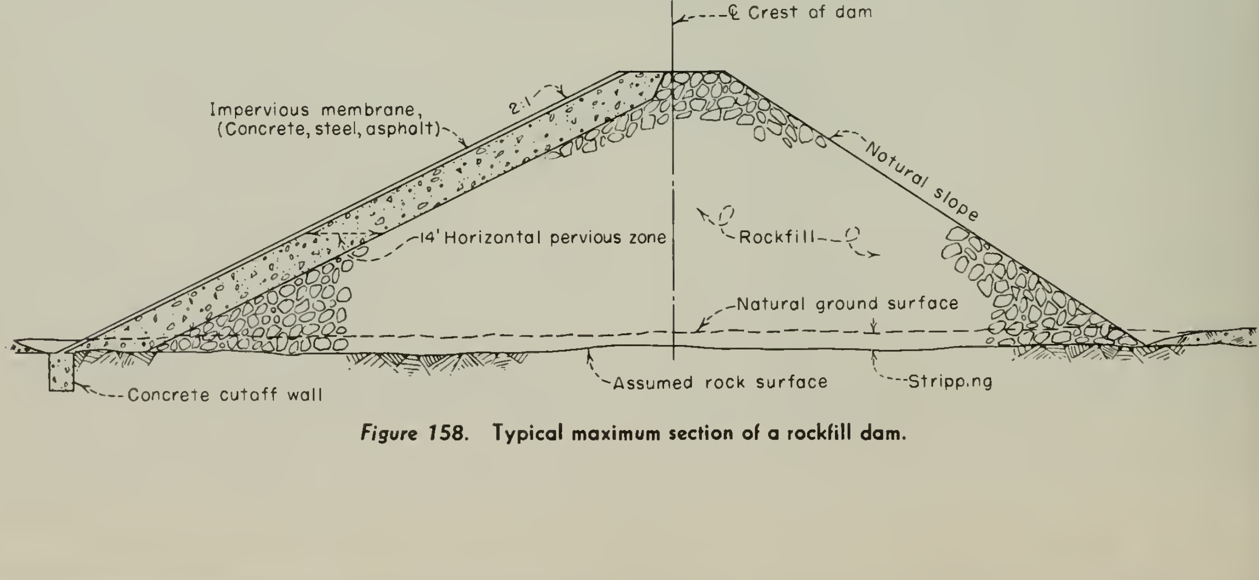


Figure 158. **Typical maximum section of a rockfill dam.**

fill during placement to help in the compaction. For small rockfill dams, however, placement of rock in relatively thin layers is considered to be a preferable method. The rock should be dumped on the embankment and spread in layers with a maximum thickness of 3 feet. The spreading operation will assure a minimum number of large voids and provide a compact rockfill. It is often desirable to sluice each layer during placement with water from high-velocity nozzles, using a volume of water equal to two or three times the volume of rock. Sluicing of quarry-run rock provides point bearing of the larger sizes, as all smaller sizes are washed into the voids. This will provide a dense fill and minimize future settlement. Sometimes gravel is sluiced into the rockfill as layers are placed.

151. ***Preparation of Upstream Face.***—Rubble masonry has been used as an upstream facing under the impervious membrane on many of the existing rockfill dams. When placed with care and with the voids properly chinked with rock spalls, rubble masonry presents a smooth, compact bedding for any type of impervious membrane. However, on low dams where only low to moderate stresses will be present, this type of facing is considered unnecessary and uneconomical. On these structures a zone of graded sand and gravel or quarry fines may be substituted for rubble masonry. This zone should have a minimum horizontal width of 14 feet to facilitate compaction. It should be constructed in 12-inch layers, thoroughly wetted and compacted by a crawler-type tractor as described in the sample specifications, appendix E. The material used in this zone should be pervious and well graded from $\frac{1}{4}$ inch to 3 inches. After placing, the upstream face can be dressed smooth to accept any type of membrane. (See sec. 145.)

D. DESIGN OF UPSTREAM FACING

152. ***Reinforced Concrete.***—The most common impervious membrane used to face rockfill dams is reinforced concrete. For low dams, a reinforced concrete slab with a minimum thickness of 8 inches should be provided. Because of the low reservoir head and the small amount of settlement to be expected, horizontal or vertical expansion joints normally are not required in the facings for low dams. However, vertical joints may be required to compensate for horizontal expansion on low dams of considerable length. These joints may be desirable for construction purposes also. Reinforcement should be provided; areas of steel equal to 0.5 percent and 0.7 percent of the concrete area, vertically and horizontally, respectively, are considered good practice. Dense, durable concrete is needed to guard against seepage and damage of the concrete due to wave action and weathering. The design of suitable concrete mixes is discussed in appendix F.

153. ***Asphaltic Concrete.***—Asphaltic concrete facing has been used on a recent (1957) rockfill dam. In the construction of Montgomery Dam [2, 3], the upstream face was sprayed to obtain penetration of the asphalt and to form a base for the hot mix. Three layers of hot asphalt mix, each 4 inches thick, were then placed on the face. The asphalt was placed by use of a standard paving machine lowered from the crest by cables. The hot mix contained 8 percent asphalt by dry weight, and gradation of the mix aggregate ranged from 11 percent passing the No. 200 screen to a maximum size of $1\frac{1}{2}$ inches.

154. ***Steel.***—Steel facing has been used on some rockfill dams [4, 5, 6]. The steel plates, $\frac{1}{4}$- to $\frac{3}{8}$-inch thick and in sizes which could be handled with the equipment available, were bolted or welded into place. The steel plate was embedded in a concrete cutoff wall at the foundation to insure a tight contact and reduce the possibility of leakage. On long dams, vertical contraction joints at approximately 25 feet, constructed of steel V-shaped troughs, are used to compensate for horizontal expansion.

155. ***Timber Planking.***—Timber planking has been used as a temporary type of membrane, but it is not recommended for general use, although it is often the cheapest type of membrane to construct. The principal objections to this type of construction are the danger of loss by fire at low

484556 O—60——17

water and the relatively short life of timber construction when alternately exposed to wetting and drying.

In construction, the timber planks, either in single or double layers, are spiked to heavy timber sills which are buried in the upstream face of the dam. Floating of the deck is prevented by fastening the sills to posts buried deep in the dam. The spacing of the sills depends upon the strength of the deck planks, which should be designed to carry the full water pressure on the span between the sills without support from the dam. At the base of the slope, the deck rests on a heavy longitudinal sill which is secured to the foundation cutoff wall. All timber should be pressure creosoted to extend its life.

E. BIBLIOGRAPHY

156. ***Bibliography.***

[1] Galloway, J. D., "The Design of Rockfill Dams," Trans. ASCE, No. 104, pt. 2, 1939, p. 1.

[2] "Asphalt Deck is Dams Watertight Layer," Engineering News-Record, December 5, 1957, p. 36.

[3] "Abrasive Rock, Asphalt Facing for Water Supply Dam," Engineering News-Record, November 15, 1956, p. 44.

[4] Hovey, O. E., "Steel Dams," American Institute of Steel Construction, New York, 1935, p. 79.

[5] Reid, H. I., "Steel Plates With Welded Joints Seal Rockfill Dam," Engineering News-Record, vol. 108, p. 761.

[6] Seger, C. P., "Steel Used Extensively in Building El Vado Dam," Engineering News-Record, vol. 115, p. 211.

Concrete Gravity Dams

A. T. LEWIS, J. S. CONRAD, AND E. L. WATSON [1]

A. INTRODUCTION

157. ***Origin and Development.***—A concrete gravity dam is a structure proportioned so that its own weight resists the forces exerted upon it. If it is constructed on an adequate foundation, a solid concrete dam is a permanent structure which requires little maintenance.

The most ancient gravity dam of record was built in Egypt more than 4,000 years B.C. of uncemented masonry. Archeological experts believe this dam was kept in perfect condition for more than 45 centuries. While the proportions of this dam, base width to height, are not known, there is evidence from the ruins of other dams of that era which indicates base widths as great as four times the height. By improved design and construction methods, the Romans were able to reduce this ratio to 3 : 1.

Uncemented masonry was not suitable for the construction of other than low dams, and other methods of construction evolved. According to records, clay mortar was first used to bind the masonry together; later lime mortar was discovered and used. The masonry type of dam was largely superseded by the cyclopean type of concrete construction, which was a forerunner of the modern concrete gravity dam. Innumerable innovations in design and construction, such as refrigeration of the mass to disperse heat of hydration, use of fly ash, separate block construction, and many others, have made possible construction of monumental structures such as Hoover Dam, 726 feet high; Grand Coulee Dam which contains more than 11 million cubic yards of concrete; and Grand Dixence Dam which is now (1958) under construction in Switzerland and which will have a completed height of 922 feet. The ratio of base width to height of all these structures is considerably less than 1 : 1.

158. ***Scope of Discussion.***—This chapter is limited to discussions of "small" concrete gravity dams; that is, structures not more than 50 feet high if on rock foundations, and with maximum net head (headwater to tailwater) not exceeding 20 feet if on pervious foundations. Unless these dams are of unusual length, it is believed that the design methods discussed herein will result in economical structures. More precise design methods may be warranted for dams which, because of their length, require a large volume of concrete. Examples of concrete dams within the scope of this discussion are Willwood Diversion Dam on the Shoshone River in Wyoming, shown in figure 159, and Woodston Diversion Dam on the South Fork of the Solomon River in Kansas, shown in figure 160.

This chapter discusses primarily stabilizing and nonstabilizing forces which act on concrete gravity dams and the requirements for stability. Additional considerations in connection with concrete structures on pervious foundation are presented and, finally, current practices regarding miscellaneous details of design or layout are briefly described. On a small dam very little economical advantage can be gained by cooling the concrete and grouting contraction joints in order that the entire structure may be analyzed as a single mass. For this reason, only the gravity method of analysis is presented in this text.

[1] Engineers, Concrete Dams Section, Bureau of Reclamation.

Figure 159. Willwood Diversion Dam, a concrete-gravity structure on the Shoshone River in Wyoming. 2G–1–9.

Figure 160. Construction view of overflow section of Woodston Diversion Dam, a concrete-gravity structure on the South Fork of the Solomon River in Kansas. 796–701–2944.

B. FORCES ACTING ON THE DAM

159. *General.* —For the design of gravity dams, it is necessary to determine the forces which may be expected to affect the stability of the structure. The forces which must be considered for gravity dams within the range of this text are those due to (1) water pressure, both external and internal (or uplift), (2) silt pressure, (3) ice pressure, (4) earthquake, (5) weight of the structure, and (6) the resulting reaction of the foundation. In designing the crest of an overflow section, the possibility of subatmospheric pressures developing between the overflowing sheet of water and the concrete should be considered (see ch. VIII).

Other forces, including wind and waves, are negligible for small dams and need not be considered in the stability analysis.

Figure 161(A) shows the normal loading conditions used in this chapter. Symbols and definitions for the normal loading conditions are given below:

ψ = angle between face of element and the vertical.

T = horizontal distance from upstream edge to downstream edge of section.

I = moment of inertia of base of section 1-foot wide about its center of gravity, equal to $\frac{T^3}{12}$.

w_c = unit weight of concrete.

w = unit weight of water.

h or h' = vertical distance from reservoir water or tailwater, respectively, to base of section.

p or p' = reservoir water or tailwater pressure, respectively, at base of section. It is equal to wh or wh'.

W_o = dead load weight above base of section under consideration including the weight of the concrete, W_c, plus such appurtenances as gates and bridges.

W_w or W_w' = vertical component of reservoir water or tailwater load, respectively, on face above base of section.

M_o = moment of W_o about center of gravity of base of section.

M_w or M_w' = moment of W_w or W_w' about center of gravity of base of section.

V or V' = horizontal component of reservoir water or tailwater load, respectively, on face above base of section. This is equal to $\frac{wh^2}{2}$ for V and $\frac{w(h')^2}{2}$ for V' for normal conditions.

M_p or M_p' = moment of V or V' about center of gravity of base of section, equal to $\frac{wh^3}{6}$ for M_p and $\frac{w(h')^3}{6}$ for M_p'.

ΣW = resultant vertical force above base of section, equal to $W_o + W_w + W_w'$.

ΣV = resultant horizontal force above base of section, equal to $V - V'$.

ΣM = resultant moment of forces above base of section about center of gravity of base of section. It is equal to $M_o + M_w - M_w' - (M_p - M_p')$.

e = distance from center of gravity of base of section to point where resultant of ΣW and ΣV intersects base of section. It is equal to $\Sigma M / \Sigma W$.

U = total uplift force on horizontal section, equal to $T\left(\frac{p+p'}{2}\right)$.

In addition to the normal loading conditions, it may be necessary to apply ice, silt, and earthquake loads. It is not likely, however, that all of these additional loads will occur at the same time. Whether these additional loads should be considered and in what combinations, should be determined by an engineer experienced in the design of dams. Silt, ice, and earthquake loads are discussed in sections 161, 162, and 163, respectively.

160. *Water Pressure.* —(a) *External.*—The external water pressure acting on a nonoverflow dam is illustrated in figure 161(A). On the upstream face, for example, the horizontal force is V and the vertical force is W_w. The weight of water is generally accepted as 62.5 pounds per cubic foot.

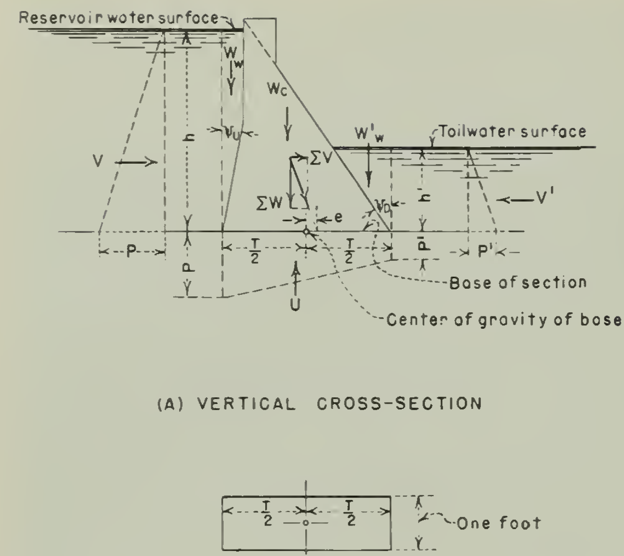


Figure 161. **Forces acting on a concrete-gravity dam.**

On overflow dams without control features, the total horizontal water pressure on the upstream face is represented by the trapezoid (*abcd*) in figure 162 in which the unit pressures at the top and at the bottom are $62.5h_1$ and $62.5h$, respectively. The line of action of the force passes through the center of gravity of the trapezoid. The vertical component of the water flowing over the top of the spillway is not used in the analysis because the water approaches spouting velocity, which greatly reduces the vertical pressure on the dam. Likewise, because of its high velocity, the stream of water on the downstream face does not exert enough pressure on the dam to warrant consideration. Where tailwater or backwater pressure exists on the downstream side, it is treated in the same manner as the tailwater pressure in figure 161(A). If gates, flashboards, or other control features are used on the crest, they are treated as part of the dam so far as the application of water pressure is concerned.

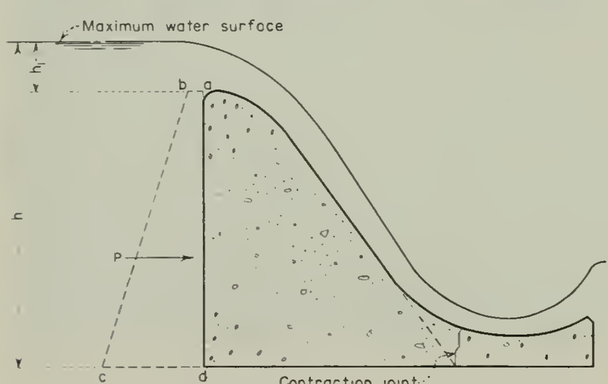


Figure 162. **Water pressures acting on an overflow concrete dam.**

(b) *Internal or Uplift Pressure.*—Uplift forces occur as internal pressures in pores, cracks, and seams in both the dam and dam foundation. It is evident that these spaces in the dam or foundation will be filled with water, which exerts a pressure in all directions. This pressure may have an important effect on the stability of the dam and must be included in the analysis. It is assumed that uplift pressures are not affected by earthquake forces.

(1) *Dams on rock foundations.*—The intensity of uplift pressure under a concrete dam on a rock foundation is difficult to determine. It is generally assumed that pore pressures in rock or concrete are effective over the entire base of the section. It is evident that under sustained loading the intensity of uplift at the upstream face is equal to the full reservoir pressure and approaches a straight-line variation from this point to tailwater pressure, or zero, at the downstream face if there is no tailwater. This is true not only at the contact between the dam and the foundation but within the body of the dam itself.

Uplift pressures can be reduced by forming drains through the concrete of the dam and by drilling drainage holes into the foundation rock. Such drains are usually provided near the upstream face of the dam although care must be exercised to insure that direct piping from the reservoir will not result. Drains of this type are provided in all Bureau of Reclamation dams of considerable height, and actual measurements of uplift under the dams have proved them to be very effective. If the rock of the foundation of a proposed dam were absolutely homogeneous, the effectiveness of the drains could be predetermined. However, owing to the presence of seams and fissures and the uncertainty of intercepting them with the drains, the safest course is to assume the straight-line variation from headwater to tailwater pressures as a measure of uplift. Any other assumptions should be verified by electric analogy or other comparable methods of analysis conducted by engineers experienced in this field.

Other methods used to reduce the uplift at the

contact of the dam with the foundation include construction of cutoff walls under the upstream face, construction of drainage channels (usually of sewer pipe) between the dam and the foundation, and pressure grouting the foundation. These methods usually are considered only as additional safety factors in the design of small dams and are not considered as justification for reducing design requirements.

(2) *Dams on pervious foundations.*—Uplift pressures under a concrete dam on a pervious foundation are related to seepage through permeable materials. Water percolating through the materials is retarded by frictional resistance, somewhat the same as water flowing through a pipe. The intensity of the uplift can be controlled by construction of properly placed aprons, cutoffs, and other devices. A discussion of these and methods for determining uplift pressures is included in sections 171 through 174.

161. *Silt Pressure.*—Nearly all streams carry an appreciable amount of silt during both normal and floodflows. Where silt is present in a stream on which a dam is built, it will eventually find its way to the reservoir and be deposited in the still water above the dam. Methods for determining the amount of silt and its deposition in a reservoir are discussed in chapter I. If allowed to accumulate against the upstream face of the dam, the silt will exert loads greater than hydrostatic pressure. In the absence of reliable test data, a rather common assumption of the magnitude of silt pressure is to consider the horizontal load as equivalent to that of a fluid weighing 85 pounds per cubic foot and the vertical weight as 120 pounds per cubic foot.

Sluiceways are often provided in gravity dams to prevent silt from accumulating in the reservoir. In diversion dams the main function of the sluiceway is to keep the headworks and canal free from silt, but some benefit may also be derived for the dam by reducing the silt load.

Many gravity dams have been designed without regard to silt load. In general, the silt load against storage dams will be a small factor but against diversion dams it is likely to be more important. In either case there is some basis for neglecting the silt load. Initially the silt load is not present, and by the time it might become a significant factor, the silt has consolidated to some extent and therefore acts less like a fluid. Furthermore, silt deposited in the reservoir will probably be somewhat impervious and will help to minimize the uplift under the dam.

Greater importance must be attributed to silt load when the primary purpose of the dam is silt retention. In this situation the designer may not be satisfied to assume an arbitrary equivalent fluid pressure as mentioned above. More precise computations of silt load can be made by combining the hydrostatic pressure with the horizontal component of the silt load, V, as determined by the Rankine formula neglecting cohesion:

$$V=\frac{w_s h^2}{2}\left(\frac{1-\sin\phi}{1+\sin\phi}\right) \qquad (1)$$

where:

w_s=submerged weight of material,

h=depth of material, and

ϕ=angle of internal friction.

Here too obstacles will be met in making assumptions regarding submerged weight, angle of internal friction, etc., owing to lack of data. A comprehensive discussion of the Rankine formula and the assumptions involved in its application is beyond the scope of this work, but it may be found in many standard texts.

162. *Ice Pressure.*—Ice pressure is produced by thermal expansion in the ice sheet and by wind drag. The necessary allowance to be made for ice load in the design of a concrete dam is difficult to determine. Data concerning the physical characteristics of ice such as its crushing strength, its modulus of elasticity, and the effects of plastic flow are inadequate and approximate. Furthermore, the thrust exerted by expanding ice depends on the thickness of the sheet, the rate of temperature rise in the ice, fluctuations in the water surface, character of the reservoir shores, slope of the upstream face of the dam, wind drag, and other factors. The rate of temperature rise in the ice is a function of rate of rise of the air temperature and the amount of snow cover on the ice. Lateral restraint of the ice sheet depends on the character of the reservoir shores and the slopes of the upstream face of the dam.

In view of all of these variables, the designer is faced with a difficult task in estimating the amount of ice pressure acting against a structure. However, several aids are available to help in the

design. In 1947 Rose [1] [2] collected and put into usable form all the known data on ice pressures. His charts, figure 163, show the thrust in kips for thicknesses up to 4 feet for air temperature rises of 5°, 10°, or 15° F. per hour (curves A, B, and C), respectively. The designer may elect to include or disregard the effects of lateral restraint and solar energy.

Since 1947 the Bureau of Reclamation has conducted field and laboratory tests on ice temperatures and pressures. These tests have been summarized by Monfore [2]. With the installed equipment, ice pressures were obtained directly from gages or by computations based on ice temperature readings. Close agreement was obtained by the two methods. In the field tests at Eleven Mile Canyon Reservoir, it was found that most of the expansion in the ice sheet took place in the upper portions and, therefore, the thrust was largely concentrated there. In the computations for thrust exerted by an 18-inch-thick sheet, it was found that no change in temperature occurred at the bottom of the ice; hence, zero pressure was assumed there. It was also demonstrated that a 5-inch snow covering had a remarkable insulating effect in preventing temperature rise and thus minimizing the ice thrust.

Within the United States it is only in the northern and mountainous regions that ice pressures can be expected to be a significant load on small dams. Where a gravity diversion dam has gates, it is common practice to heat the gates and thus prevent ice forming against the metal, but in the abutment sections ice load can be an important consideration.

163. *Earthquake.*—(a) *General.*—Earthquakes impart accelerations to the dam which may increase the water and silt pressures on the dam and the stresses within the dam. Some allowance for earthquake loads must be made in the design of concrete gravity dams to be constructed in seismic zones. In addition to the increase in water loads and in silt loads (if applicable), the effect of earthquake on the dead load of the structure must be recognized.

Both vertical and horizontal earthquake loads should be applied in the direction which produces the least stable structure. For the condition of full reservoir this will be a foundation shock in the upstream direction and a foundation shock downward. The first increases the water load and produces an overturning moment due to inertia on the concrete. The second, in effect, causes the concrete and water above a sloping face to weigh less and in this way reduces the stability of the structure.

In order to determine the total forces due to an

[2] Numbers in brackets refer to items in the bibliography, sec. 178.

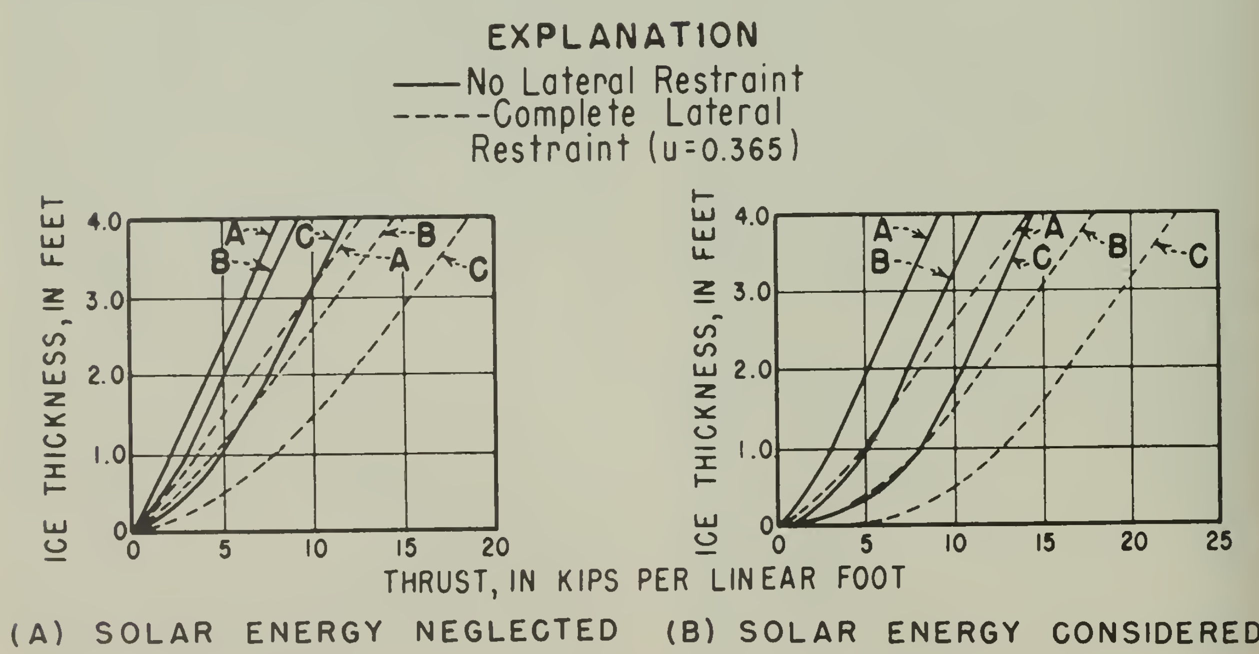


Figure 163. Ice thrust in relation to ice thickness, air-temperature rise, and restraint.

earthquake, it is necessary to establish the earthquake intensity or acceleration. This is usually expressed in relation to the acceleration due to gravity. The accelerations that may be reasonably expected at the damsite are determined from a consideration of the geology of the site, proximity to major faults, previous earthquake history of the region, and such seismic records as are available. In areas not subjected to extreme earthquake conditions a horizontal acceleration of 0.10 of gravity and a vertical acceleration of 0.05 of gravity are generally used.

Experimental and analytical procedures show that, because of the internal shear resistance of the silt, an earthquake acceleration up to approximately 0.30 of gravity is only about half as effective in silt as in water. Since the unit weight of water is about one-half that of silt, the increase in pressure on the dam due to earthquake is approximately the same for either silt or water.

Resonance in low dams is not likely to occur during earthquake shock for several reasons. The fundamental period of vibration of a concrete dam 50 feet high and of triangular cross section is is between 0.03 and 0.04 of a second. Vibration periods of important earth shocks are estimated by various authorities to lie between 0.2 and 1.0 second; therefore, serious resonance between the dam and the earth shock will not take place. Furthermore, earthquakes are experimentally and analytically treated as harmonic but ground motions recorded in the destructive zone of the quake do not appear to be harmonic. Also, many forms of damping that are difficult to evaluate act to prevent resonance.

In 1952 Zanger [3] presented formulas for computing the hydrodynamic pressures exerted on vertical and sloping faces by horizontal earthquake. The formulas were derived by electric analogy, based on the assumption that water is incompressible. For low dams the error involved in computing the earthquake force on the water because of this simplifying assumption is probably less than 1 percent.

(b) *Horizontal Earthquake.*—The effect of inertia on the concrete should be applied at the center of gravity of the mass, regardless of the shape of the cross section. For dams with vertical or sloping upstream faces, the increase in water pressure, P_e, in pounds per square foot, at any elevation due to horizontal earthquake is given by the following equation:

$$P_e = C\lambda wh \tag{2}$$

where:

C (a dimensionless coefficient giving the distribution and magnitude of pressures)

$$= \frac{C_m}{2}\left[\frac{y}{h}\left(2-\frac{y}{h}\right)+\sqrt{\frac{y}{h}\left(2-\frac{y}{h}\right)}\right]$$

λ = the earthquake intensity = $\frac{\text{earthquake acceleration,}}{\text{acceleration of gravity}}$

w = unit weight of water, pounds per cubic foot,

h = total depth of reservoir at section being studied, feet,

y = the vertical distance from the reservoir surface to the elevation in question, feet, and

C_m = maximum value of C for a given constant slope (fig. 164).

Values of C for various degrees of slope and relations of y and h may be obtained from figure 165. It may be shown analytically that the total horizontal force, V_e, above any elevation y distance below the reservoir surface, and the total overturning moment, M_e, above that elevation are:

$$V_e = 0.726\ P_e y \tag{3}$$

and

$$M_e = 0.299\ P_e y^2 \tag{4}$$

For dams with a combination vertical and sloping face, the procedure to be used is governed by

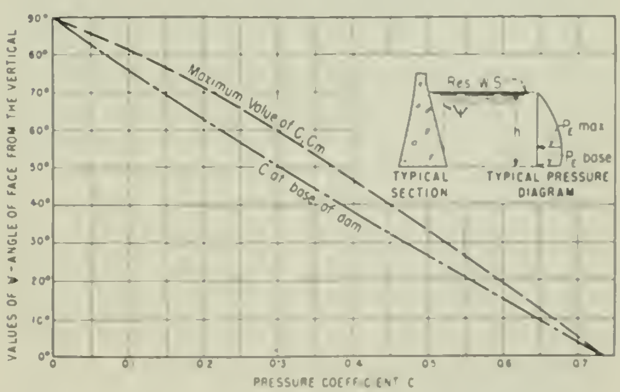
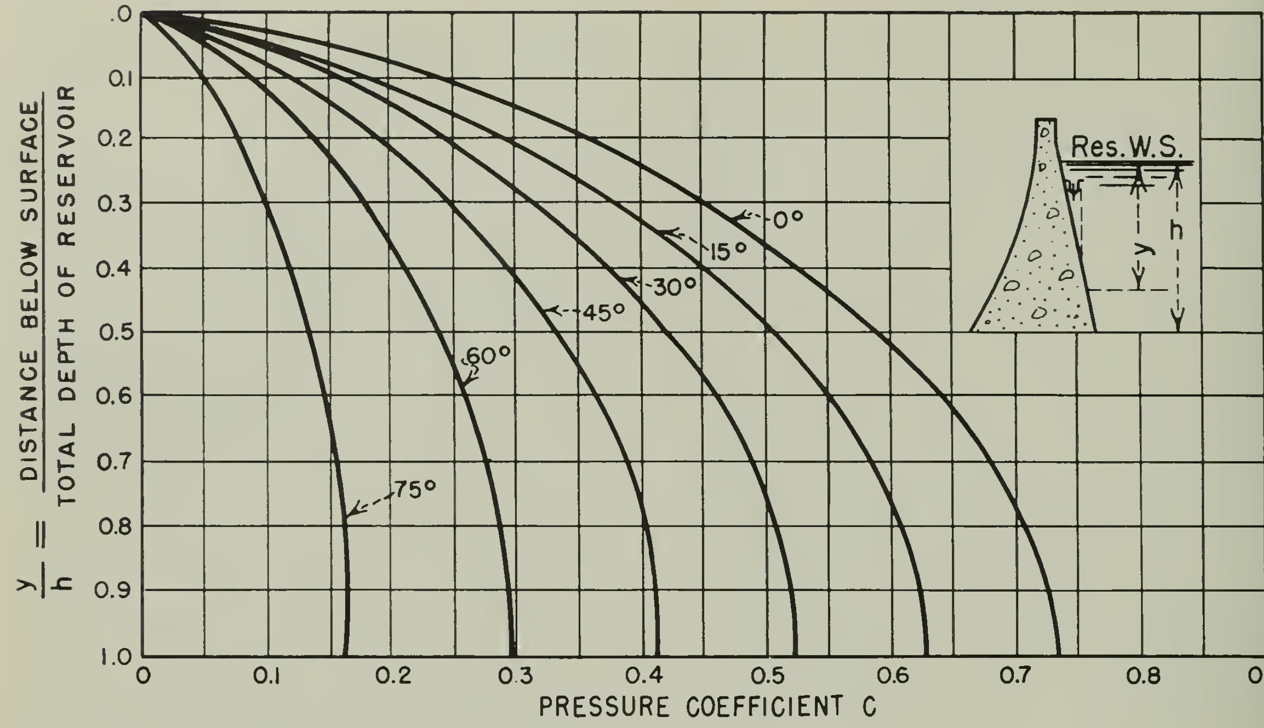


Figure 164. Base and maximum pressure coefficients for constant sloping faces.



$$C=\frac{1}{2}C_m\left[\frac{y}{h}\left(2-\frac{y}{h}\right)+\sqrt{\frac{y}{h}\left(2-\frac{y}{h}\right)}\right]$$

where: C_m is maximum C value from figure 164

***Figure 165.* Coefficients for pressure distribution for constant sloping faces.**

the relation of the height of the vertical portion to the total height of the dam, as follows:

(1) If the height of the vertical portion of the upstream face of the dam is equal to or greater than one-half the total height of the dam, analyze as if vertical throughout.

(2) If the height of the vertical portion of the upstream face of the dam is less than one-half of the total height of the dam, use the pressures on a sloping line connecting the point of intersection of the upstream face of the dam and reservoir surface with the point of intersection of the upstream face of the dam and the foundation.

(c) *Vertical Earthquake.*—On sloping faces of dams the weight of the water above the slope should be modified by the appropriate acceleration factor. The weight of the concrete also should be modified by this acceleration factor.

164. *Weight of Structure.*—The weight of the structure includes the weight of the concrete plus appurtenances such as gates and bridges. However, for most low dams only the dead load due to the weight of the concrete is used in the analysis. The unit weight of concrete ordinarily is considered to be 150 pounds per cubic foot. The total weight acts vertically through the center of gravity of the cross section.

165. *Reaction of Foundation.*—Under stable conditions the resultant of the horizontal and vertical loads on the dam will be balanced by an equal and opposite force which constitutes the reaction of the foundation. The vertical reaction of the foundation, computed without uplift, is represented by the trapezoid $A12B$, figure 166(B).

The stresses $A1$ and $B2$ are determined by the use of eccentric loading formulas and are as follows:

$$A1=\frac{\Sigma W}{T}\left(1-\frac{6e}{T}\right) \tag{5}$$

$$B2=\frac{\Sigma W}{T}\left(1+\frac{6e}{T}\right) \tag{6}$$

When uplift is introduced and the uplift pressure at the upstream face is less than $A1$, the dam is stable against overturning and the final stresses may be computed by the above formulas. If the uplift pressure at the upstream face is greater than $A1$, the foundation pressure at the base will have to be revised as in figure 166(D)).

To revise the foundation pressure due to excessive uplift, the following procedure should be used:

(1) A horizontal crack is assumed to exist and extend from the upstream face toward the downstream face to a point where the vertical stress of the adjusted diagram is equal to the uplift pressure at the upstream face (fig. 166(D).)

(2) From figure 166(A) and (D) and taking moments about the center of gravity of the base, there are obtained:

$$e'=\frac{\Sigma M}{\Sigma W-A3\cdot T} \tag{7}$$

$$T_1=3\left(\frac{T}{2}-e'\right) \tag{8}$$

and

$$B5=\frac{2(\Sigma W-A3\cdot T)}{T_1}+A3 \tag{9}$$

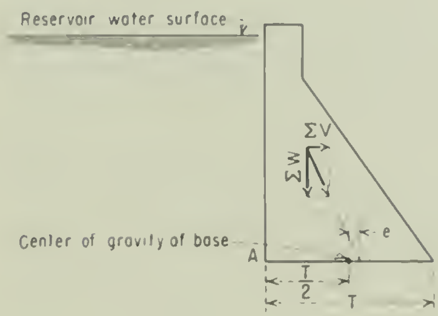
C. REQUIREMENTS FOR STABILITY

166. *General.*—A concrete gravity dam must be designed to resist, with ample factor of safety, these three tendencies to destruction: (1) overturning, (2) sliding, and (3) overstressing.

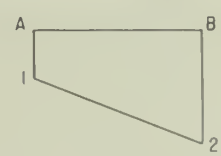
167. *Overturning.*—There is a tendency for a gravity dam to overturn about the downstream toe at the foundation, or about the downstream edge of any horizontal section. If the vertical stress at the upstream edge of any horizontal section computed without uplift exceeds the uplift pressure at that point, the dam is considered to be safe against overturning with an ample factor of safety. If the uplift pressure at the upstream face exceeds the vertical stress at any horizontal section computed without uplift, the uplift forces along the assumed horizontal crack greatly increase the tendency for the dam to overturn about the downstream face. Under this condition, however, if $B5$ in figure 166(D) is less than the allowable stress in the concrete and the allowable stress in the foundation, the dam is considered to be safe against overturning.

168. *Sliding.*—The horizontal force, ΣV, tends to displace the dam in a horizontal direction. This tendency is resisted by the frictional and shearing resistance of the concrete or the foundation.

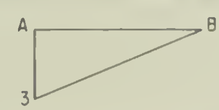
The shear friction factor [4], a criterion normally used for higher dams, is not recommended for use in the design of dams within the scope of this text although it is recognized that economical design of concrete dams on good rock may be penalized thereby. Cohesive characteristics of the concrete or rock, which greatly affect the shear friction



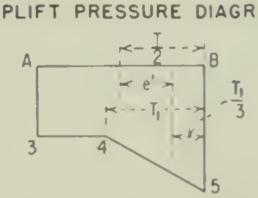
(A) VERTICAL CROSS-SECTION



(B) BASE PRESSURE DIAGRAM WITHOUT UPLIFT



(C) UPLIFT PRESSURE DIAGRAM



(D) COMBINED BASE PRESSURE AND UPLIFT PRESSURE DIAGRAM

Figure 166. Foundation base pressures for a concrete-gravity dam.

factor, must be determined by special laboratory tests or estimated by an engineer who has had considerable experience in this specific field. For small structures where it is not economical to perform these tests or to obtain expert advice, the usual method of checking the structure for horizontal displacement is by determination of a sliding factor.

The allowable sliding factor is the coefficient of static friction between two sliding surfaces reduced by an appropriate factor of safety. If f represents the allowable sliding factor, a dam is considered safe against sliding when $\frac{\Sigma V}{\Sigma W-U}$ is equal to or less than f. Exact values for coefficients of static friction cannot be determined without the benefit of laboratory tests, but the allowable sliding factors given below, which include ample factors of safety for concrete against sliding on the various foundation materials, may be used as a general guide:

Material:	f
Sound rock, clean and irregular surface	0. 8
Rock, some jointing and laminations	. 7
Gravel and coarse sand	. 4
Sand	. 3
Shale	. 3
Silt and clay	([1])

[1] Testing required.

Concrete cutoff walls are often provided on structures constructed on foundations other than rock. The cutoff, properly proportioned and reinforced, prevents displacement of the structure by internal shear resistance of the cutoff itself and the additional volume of soil that must be moved before the structure can slide. To accomplish this objective, the cutoff must be designed as a cantilever beam loaded with the horizontal force that is in excess of the foundation's resistance to sliding.

If a stratum of a soil weaker than the overlying strata exists in the foundation, the sliding should also be investigated along the top of the weak bed. In this case, however, the weight of the overlying strata and the shearing resistance of materials downstream from the structure also would be considered in computing the sliding factor.

169. *Overstressing.*—The unit stresses in the concrete and the foundation must be kept within prescribed maximum values. Normally, the stresses in the concrete of gravity dams within the scope of this text will be so low that a concrete mix designed as specified in appendix F to meet other requirements such as durability and workability will attain sufficient strength to insure a factor of safety of at least 4 against overstressing.

The foundation should be investigated and the maximum allowable stress established. Engineering properties of foundation materials and accompanying considerations affecting such properties are discussed in chapter IV. Local codes of allowable bearing pressures and engineers qualified in evaluating the adequacy of foundation materials should be consulted as far as possible before final design. Suggested allowable bearing values for footings for structures appurtenant to small dams are given in appendix C. These may be used as a guide in designing small concrete dams. If there is any doubt as to the proper classification and adequacy of the foundation materials, laboratory tests should be made to determine the allowable bearing pressures.

D. DAMS ON PERVIOUS FOUNDATIONS

170. *General.*—Small gravity dams constructed on rock present relatively few difficult foundation problems. The design of dams on pervious foundations, however, involves problems of erosion of the foundation material and seepage under the structure. The complexity of these problems varies greatly and depends on the type, stratification, permeability, homogeneity, and other properties of the foundation materials as well as the size and physical requirements of the structure itself.

Concrete gravity storage dams and diversion dams more than 20 feet high on pervious foundations usually require extensive field and laboratory investigations. Such structures are beyond the scope of this text, which for pervious foundations is limited to gravity dams whose maximum net head (headwater to tailwater) is not appreciably greater than 20 feet.

The control of erosion, seepage, and uplift forces under dams constructed on pervious

foundation often requires the use of some, all, or various combinations of the following devices:

(1) Upstream apron, with or without cutoffs at the upstream end.

(2) Downstream apron, with or without cutoffs at the downstream end, and with or without filters and drains under the apron.

(3) Cutoffs at the upstream or downstream end or at both ends of the weir or control section, with or without filters or drains under the section.

The locations and extent of these devices to obtain optimum safety and economy of design depend on many conditions. Therefore, only brief discussions of devices or combinations of devices which might be used with relatively simple diversion dams are discussed.

171. ***Aprons.***—A concrete apron may be placed upstream of the dam in conjunction with one of the various types of cutoff walls. The function of the apron is to increase the length of the path of percolation in order to reduce uplift under the main portion of the dam. Usually the apron is connected to the dam and to a concrete cap over the piling with flexible waterstops, which allow differential movement to take place without accompanying detrimental cracking. The safety of the structure may be further improved by placing an impervious earth blanket over a portion of the concrete apron and the streambed upstream from it.

Downstream concrete aprons have two functions. They lengthen the path of percolation in the foundations and also provide a basin where the energy of the overflowing water may be safely dissipated. Energy dissipation on the concrete helps to prevent dangerous erosion at the toe of the dam. In cases where it is not feasible to construct a concrete apron of sufficient length to avoid erosion completely, additional protection may be gained by placing riprap downstream from the apron.

172. ***Cutoff Walls.*** Cutoff walls may be constructed of timber, concrete, cement-bound curtains, steel sheet piling, or impervious earth compacted in a trench. Each type can be effective under appropriate circumstances.

Timber piling may be used as cutoffs under upstream or downstream aprons. Dimension timber piling is not recommended where driving is necessary. Better construction practice consists of erecting overlapping treated timbers in an excavated open trench and then backfilling and compacting impervious material in the trench around the timbers. This method prevents brooming or splitting of the timbers which results when timber is driven through sand and gravel. If the piling is seated on an impervious stratum and is properly connected to the concrete apron, a tight barrier to underseepage is provided.

Concrete cutoffs may be used under aprons or under the weir section. They may be constructed by trenching with a machine or by hand labor and backfilling against the undisturbed sides of the trench, or they may be constructed by forming the concrete wall in an open excavated trench and then backfilling and compacting impervious material in the trench and around the wall. A concrete cutoff is probably the best type of wall for preventing underseepage and is often used. In addition to acting as a cutoff, such a cutoff can be designed to contribute substantially to the stability (sliding resistance) of the dam when placed under the weir section.

Cement-bound curtain cutoffs have been used under the upstream apron on two concrete gravity diversion structures by the Bureau of Reclamation. The construction of this type of cutoff is described in section 126(c).

Sheet piling cutoffs of interlocking steel sections are often used under the aprons of diversion dams. The main advantage of this type is that it is economical and is easily connected to a concrete apron at riverbed. A more complete discussion of sheet piling cutoffs will be found in section 126(d).

173. ***Filters and Drains.*** Relief of uplift pressures under the apron or downstream toe of the dam may be accomplished by drains. Drains are often of sewer pipe and are laid in graded material which acts as a filter. They may be perforated pipe or plain pipe laid with open joints. The drains may be located at the downstream toe of the dam, at selected places under the downstream apron, and immediately upstream from the downstream cutoff.

Weep holes are commonly used for relief of uplift pressure under aprons and behind walls. It is important that the gradation of the filter materials used in conjunction with the weep holes be carefully selected with respect to the gradation of the foundation materials to prevent piping.

Both uniform grain-size and graded filters are used. The design of filters is given in section 126 (h) and (i).

174. *Uplift and Seepage.*—Cutoff walls, aprons, and drains are installed for two reasons: To control the amount of seepage under the dam, and to limit the intensity of the uplift so that the stability of the structure will not be threatened. Several factors such as head on the dam, permeability of the foundation, length of upstream and downstream aprons, depths and tightness of cutoff, and effectiveness of drains enter into consideration of underseepage and uplift.

The magnitude and distribution of seepage forces in the foundation and the amount of underseepage for a given coefficient of permeability can be obtained from a flow net. A discussion of the flow net together with references to standard texts which treat the method of drawing and applying the flow net to problems involving subsurface flow is given in section 125(c). The construction of flow nets for analysis of underseepage problems for small diversion dams constructed on pervious foundations is not required for the reasons given in the referenced discussion.

The amount of underseepage can be approximated by use of the Darcy formula, as given in section 125(b). The weighted-creep theory, as developed by Lane [5], is suggested as a means for designing low concrete dams on pervious foundations to be safe against uplift pressures and piping. Although this is an empirical method, considerable confidence has been placed in it by many engineers and it has been successfully used for the design of many structures.

Mr. Lane gives credit for various concepts of creep analysis to the early investigators, Clibborn, Beresford, Bligh, Griffith, and others. He did, however, test his theory by analysis of more than 200 dams on pervious foundations, both failures and nonfailures. His main conclusions (omitting those regarding the "short-path" which is not applicable here) were as follows:

(1) The weighted-creep distance of a cross section of a dam is the sum of the vertical creep distances (steeper than 45°) plus one-third of the horizontal creep distances (less than 45°).

(2) The weighted-creep head ratio is the weighted-creep distance divided by the effective head.

(3) Reverse filter drains, weep holes, and pipe drains are aids to security from underseepage, and recommended safe weighted-creep head ratios may be reduced as much as 10 percent if they are used.

(4) Care must be exercised to insure that cutoffs are properly tied in at the ends so that the water will not outflank them.

(5) The upward pressure to be used in design may be estimated by assuming that the drop in pressure from headwater to tailwater along the contact line of the dam and foundation is proportional to the weighted-creep distance.

Based on his findings, Mr. Lane recommended the weighted-creep ratios shown below:

Material:	*Ratio*
Very fine sand or silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Coarse gravel including cobbles	3.0
Boulders with some cobbles and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay or hardpan	1.6

Figure 167 is an example of the application of Lane's weighted-creep theory to the design of a concrete dam. In this example, the design is investigated to determine on what types of foundations this dam would be judged safe and the magnitudes of the uplift at various points under the structures are calculated. For the purposes of this example, it is assumed that the maximum head differential occurs with the headwater at the elevation of the crest. All other combinations of headwater and tailwater should

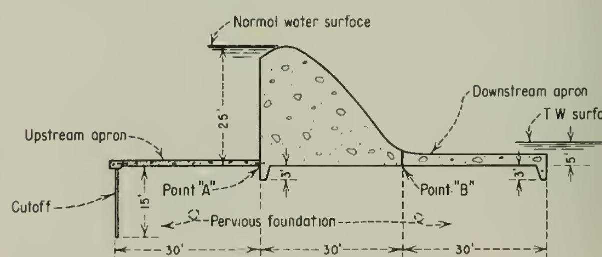


Figure 167. **Typical section of a concrete dam on a pervious foundation.**

be investigated to establish maximum loading conditions.

For figure 167:

$$\text{Weighted length of path} = 15+15+(4\times 3)$$

$$+\frac{1}{3}(30+30+30)=72 \text{ feet}$$

$$\text{Head on structure} = \text{Headwater} - \text{tailwater} = 25-5=20 \text{ feet}$$

$$\text{Weighted-creep ratio} = \frac{72}{20} = 3.6$$

According to Lane's recommended ratios, this dam would be safe on clay or on medium and coarse gravel, but not on silt, sand, or fine gravel. With properly placed drains and filters, the structure would probably be considered safe on a fine gravel foundation as indicated by conclusion (3).

The uplift may be computed as indicated by conclusion (5).

$$\text{Uplift, point A} = 20-\frac{(15+15+10)}{72}\times 20$$

$$+5 \text{ (depth of tailwater above foundation level)}$$

$$=20-11.1+5$$

$$=13.9 \text{ feet.}$$

$$\text{Uplift, point B} = 20-\frac{(15+15+10+3+3+10)}{72}\times 20+5$$

$$=20-15.6+5$$

$$=9.4 \text{ feet.}$$

$$\text{Total uplift} = \frac{(13.9+9.4)}{2}\times 62.5\times 30$$

$$=21{,}840 \text{ pounds per foot of crest length of dam.}$$

The weighted-creep head ratio can be increased by increasing the depth of cutoff or by increasing the apron length. Either of these alternatives would also decrease the uplift under the structure. The dam shown in figure 167 is a rather common installation and the example was simplified to illustrate the method. Discussion of a more complicated design such as a dam with two or more deep cutoffs, which requires application of the "short-path" theory, has been omitted. Mr. Lane's article [5] should be consulted for details for complex designs.

In addition to the flow net and weighted-creep methods of estimating the distribution of uplift pressure are Khosla's method of independent variables [6] and Rao's relaxation method [7] which can be used for making computations of uplift at critical points along the base of the structure. Because these theories are highly mathematical they are not discussed in this text.

Seepage forces and piping in a pervious foundation are discussed in section 125(c). The possibility of piping may be alleviated by several methods. As shown on figure 167, a cutoff may be constructed at the downstream end of the apron. A drain laid in graded material which acts as a filter may be placed immediately upstream of the cutoff, or riprap on a graded blanket of gravel may be placed on the material downstream of the apron to increase the downward forces.

E. DETAILS OF LAYOUT AND DESIGN

175. *General.*—If a concrete dam is appreciably more than 50 feet in length, it is necessary to divide the structure into blocks by providing transverse contraction joints. The spacing of the joints is determined by the capacity of the concreting facilities to be used and considerations of volumetric changes and attendant cracking caused by shrinkage and temperature variations. The possibilities of detrimental cracking can be greatly reduced by the selection of the proper type of cement and by careful control of mixing and placing procedures (see appendix F). In no case, however, is it advisable to exceed 50-foot spacing of contraction joints in constructing small concrete dams. Where foundation conditions are such that undesirable differential settlement or displacement between adjacent blocks will occur, shear keys are formed in the contraction joints. These may be formed vertically, horizontally, or a combination of both depending on the direc-

tion of the expected displacement. Leakage through the contraction joints is controlled by placing rubber or metal waterstops across the joints near the upstream face of the dam. Typical details of shear keys and waterstops are shown in figures 168 and 169.

176. *Nonoverflow Sections.*—The elevation of the top of a nonoverflow dam is established by assuming a safe freeboard above the maximum high water surface in the reservoir. The freeboard should be sufficient to allow for the maximum wave height as given in the first tabulation of section 136. Although only one-half of the wave height is above the mean water level, the full height is ordinarily used to allow for the run of the waves up the face of the dam. A minimum freeboard of 3 feet is recommended for most small concrete dams.

The top width is determined by such requirements as the need for travel across the dam or for access to gate-operating mechanism, and by climatic conditions. A top width of less than 4 feet is not recommended.

The width of the base and the slope of the downstream face are determined by a stability analysis. The customary method is to assume a section with the slope of the downstream face approximately

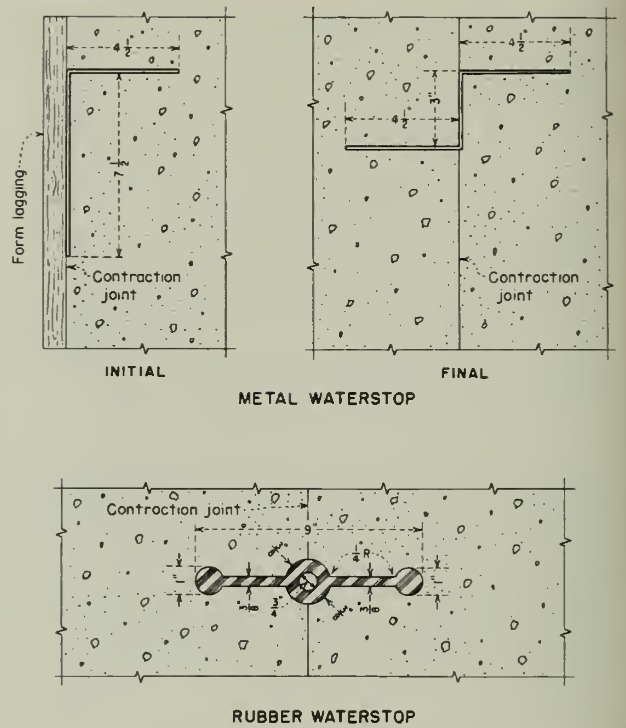


Figure 169. **Waterstop installations.**

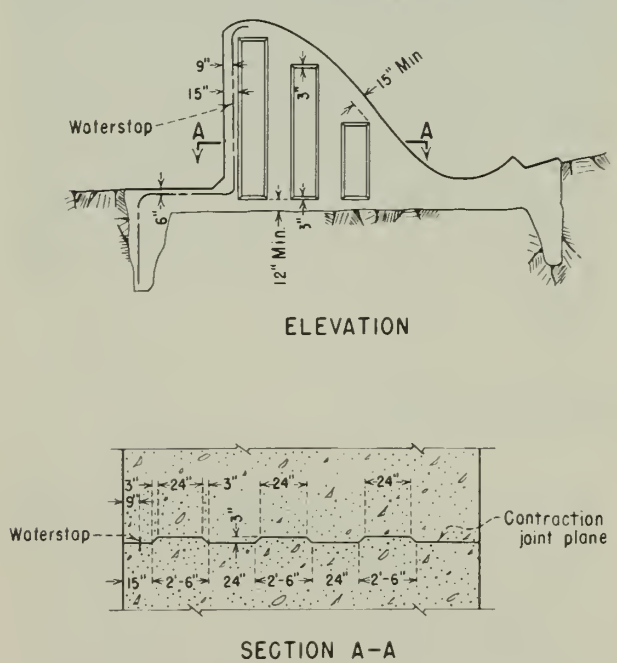


Figure 168. **Typical contraction joint shear keys.**

0.70 horizontally to 1.0 vertically and intersecting a vertical upstream face at the top of the dam. The assumed section is then analyzed and modified as required by the analysis until it meets the stability requirements. If the dam is stable about its base and also about any section where there is a break in the continuity of slope of either the upstream or downstream face, the portion of the dam between any of these sections is stable and does not require analyzing.

177. *Overflow Sections.*—In general, the method for determining the stability of overflow dams is the same as that previously described for nonoverflow dams.

The shape of the crest, the profile of the downstream face, and details of the energy dissipating basin or bucket are discussed in chapter VIII. It is customary to provide a longitudinal contraction joint at the downstream toe, as shown in figure 162, and then only that portion of the dam upstream of the joint is used in the stability computations.

In cases where the dissipating device extends only a short distance below the downstream toe

and is fairly massive, the contraction joint may be omitted. The structure below the toe is then included in the stability analysis and is so reinforced that it and the gravity portion will act as a unit. Under certain conditions an apron at the upstream face may be the most economical arrangement to insure stability. This structure, properly reinforced, is also used in the stability analysis.

Overflow dams utilizing control features introduce an additional problem. The forces acting on these features may produce tension in the upper portion of the dam which will require adequate reinforcement.

F. BIBLIOGRAPHY

178. *Bibliography.*

[1] Rose, Edwin, "Thrust Exerted by Expanding Ice Sheet," Trans. ASCE, vol. 112, 1947, p. 871.

[2] Monfore, G. E., "Experimental Investigations by the Bureau of Reclamation," Trans. ASCE, vol. 119, 1954, p. 126.

[3] Zangar, C. N., Engineering Monograph No. 11, "Hydrodynamic Pressures on Dams Due to Horizontal Earthquake Effects," Bureau of Reclamation, May 1952.

[4] Design Standards No. 2, "Treatise on Dams," Chapter 9, Gravity Dams, Bureau of Reclamation.

[5] Lane, E. W., "Security from Under-seepage," Trans ASCE, vol. 100, 1935, p. 1235.

[6] Khosla, "Design of Dams on Permeable Foundations," Central Board of Irrigation, India, September 1936.

[7] Rao, "A New Approach to the Design of Weirs on Permeable Foundations," Central Board of Irrigation, India, January 1951.

Spillways

C. J. HOFFMAN [1]

A. GENERAL

179. ***Function.***—Spillways are provided for storage and detention dams to release surplus or floodwater which cannot be contained in the allotted storage space, and at diversion dams to bypass flows exceeding those which are turned into the diversion system. Ordinarily, the excess is drawn from the top of the pool created by the dam and conveyed through an artificial waterway back to the river or to some natural drainage channel. Figure 170 shows a small spillway in operation.

The importance of a safe spillway cannot be overemphasized; many failures of dams have been caused by improperly designed spillways or by spillways of insufficient capacity. Ample capacity is of paramount importance for earthfill and rockfill dams, which are likely to be destroyed if overtopped; whereas, concrete dams may be able to withstand moderate overtopping. Usually, increase in cost is not directly proportional to increase in capacity. Very often the cost of a spillway of ample capacity will be only moderately higher than that of one which is obviously too small.

In addition to providing sufficient capacity, the spillway must be hydraulically and structurally adequate and must be located so that spillway discharges will not erode or undermine the downstream toe of the dam. The spillway's bounding surfaces must be erosion resistant to withstand the high scouring velocities created by the drop from the reservoir surface to tailwater, and usually some device will be required for dissipation of energy at the bottom of the drop.

The frequency of spillway use will be determined by the runoff characteristics of the drainage area and by the nature of the development. Ordinary riverflows are usually stored in the reservoir, diverted through headworks, or released through outlets, and the spillway is not required to function. Spillway flows will result during floods or periods of sustained high runoff when the capacities of other facilities are exceeded. Where large reservoir storage is provided, or where large outlet or diversion capacity is available, the spillway will be utilized infrequently. At diversion dams where storage space is limited and diversions are relatively small compared to normal river flows, the spillway will be used almost constantly.

180. ***Selection of Inflow Design Flood.***—(a) *General Considerations.*—When floods occur in an unobstructed stream channel, it is considered a natural event for which no individual or group assumes responsibility. However, when obstructions are placed across the channel, it becomes the responsibility of the sponsors either to make certain that hazards to downstream interests are not appreciably increased or to obligate themselves for damages resulting from operation or failure of such structures. Also, the loss of the facility and the loss of project revenue occasioned by a failure should be considered.

If danger to the structures alone were involved, the sponsors of many projects would prefer to rely on the improbability of an extreme flood occurrence rather than to incur the expense necessary to assure complete safety. However, when the risks involve downstream interests, including widespread damage and loss of life, a conservative attitude is required in the development of the inflow design flood. Consideration of potential damage should not be confined to conditions existing at the time of construction. Probable

[1] Engineer, Spillways and Outlet Works Section, Bureau of Reclamation

Figure 170. **A small chute spillway in operation at Shadow Mountain Dam on the Colorado River in Colorado.**

future development in the downstream flood plain, encroachment by farms and resorts, construction of roads and bridges, etc., should be evaluated in estimating damages and hazards to human life that would result from failure of a dam.

Dams impounding large reservoirs and built on principal rivers with high runoff potential unquestionably can be considered to be in the high-hazard category. For such developments, conservative design criteria are selected on the basis that failure cannot be tolerated because of the possible loss of life and because of the potential damages which could approach disaster proportions. However, small dams built on isolated streams in rural areas where failure would neither jeopardize human life nor create damages beyond the sponsor's financial capabilities can be considered to be in a low-hazard category. For such developments design criteria may be established on a much less conservative basis. There are numerous instances, however, where failure of dams of low heights and small storage capacities have resulted in loss of life and heavy property damage. Most small dams will require a reasonable conservatism in design, primarily because of the criterion that a dam failure must not present a serious hazard to human life.

The selection of the magnitude of an inflow design flood involves a policy decision. The following spillway capacity requirements, established by the Bureau of Reclamation for consideration of dams to be constructed by means of Federal loans under the Small Reclamation Projects Act of 1956, are offered as a guide in making such a decision:

"(1) In case that failure of the dam would increase the danger to human life, the spillway must have sufficient capacity to accommodate the maximum probable flood when routed through the reservoir.

"(2) In cases where the failure of the dam would not increase the danger to human life but would

endanger the continued operation of the responsible organization or would cause heavy property damage, plans involving a reasonable risk will be permitted if the report shows that the risk is clearly understood by the applying organization.

"(3) In case the failure of the dam would not jeopardize human life, the continued operation of the responsible organization or heavy property damage, the Bureau will not recommend against approval of the loan because of inadequate capacity but should warn the organization of the risk involved and disclaim any responsibility in case of failure."

The foregoing requirements are essentially the same as were agreed to by a number of prominent private consulting engineers in 1946.

(b) *Inflow Design Flood Hydrographs.*—Chapter II discusses the determination of floodflows which may be used as inflow design floods. The procedures presented permit the derivation of inflow flood hydrographs of three magnitudes: The maximum probable flood, the flood for assumption A conditions, and the flood for assumption B conditions (sec. 53). All of these floods are based on the hydrometeorological approach, which requires estimates of storm potential and of the amount and distribution of runoff.

Determination of the *maximum probable flood* is based on rational consideration of the chances of simultaneous occurrence of the maximum of the several elements or conditions which contribute to the flood. Such a flood is the largest that reasonably can be expected and is ordinarily accepted as the inflow design flood for dams where failure of the structure would increase the danger to human life.

The *flood for assumption A* is based on storm values less than probable maximum and on the assumption that the watershed soils are near saturation from a previous storm. In most instances these storm values represent precipitation amounts slightly greater than those which have been observed for respective locations within the United States. The magnitude of this flood is believed to be the minimum appropriate for use where the spillway capacity requirements correspond to (2) in the preceding subsection.

The *flood for assumption B* is based on storm values the same as used for the flood for assumption A, but the watershed soils are considered to be at average moisture content for the time of year when the maximum floods are likely to occur, rather than saturated. The magnitude of this flood is believed to be the minimum appropriate for use where the spillway capacity requirements correspond to (3) in the preceding subsection.

In the case of a minor structure with insignificant storage where it is permissible to anticipate failure within the useful life of the project, a 50- or 100-year frequency flood may be used as the inflow design flood. A discussion of these floods and their determination is given in sections 42 and 54. Estimates of floods of these magnitudes may also be required to establish the capacity of a service or principal spillway in those cases where an auxiliary spillway will serve to augment the smaller spillway.

181. *Relation of Surcharge Storage to Spillway Capacity.*—Streamflow is normally represented in the form of a hydrograph, which charts the rate of flow in relation to time. A typical hydrograph representing a storm runoff is illustrated in figure 171. The flow into a reservoir at any time and the momentary peak can be read from the curve. The area under the curve is the volume of the inflow, because it represents the product of rate of flow and time.

Where no storage is impounded by a dam, the spillway must be sufficiently large to pass the peak of the flood. The peak rate of inflow is then of primary interest and the total volume in the flood is of lesser importance. However, where a relatively large storage capacity above normal reservoir level can be made available economically by a higher dam, a portion of the flood volume can be retained temporarily in reservoir surcharge space and the spillway capacity can be reduced considerably. If a dam could be made sufficiently high to provide storage space to impound the entire volume

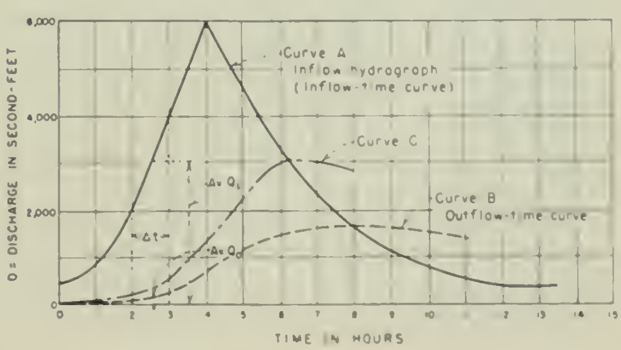


Figure 171. Inflow and outflow hydrographs.

of the flood above normal storage level, theoretically no spillway other than an emergency type would be required, provided the outlet capacity could evacuate the surcharge storage in a reasonable period of time in anticipation of a recurring flood. In such a case the maximum reservoir level would depend entirely on the volume of the flood and the rate of inflow would be of no concern. From a practical standpoint, however, there will be relatively few sites that will permit complete storage of an inflow design flood by surcharge storage. Such sites usually will be off-channel reservoirs; that is, reservoirs which are supplied by canal and which have small tributary drainage areas.

In many reservoir projects, economic considerations will necessitate a design utilizing surcharge. The most economical combination of surcharge storage and spillway capacity requires flood routing studies and economic studies of the costs of spillway-dam combinations, subsequently described. However, in making these studies, consideration must be given to the minimum size spillway which must be provided for safety. The inflow design flood hydrographs determined by the methods given in section 53 are for floods resulting from runoff from rain. Normally, such floods will have the highest peak flows but not always the largest volumes. When spillways of small capacities in relation to these inflow design flood peaks are considered, precautions must be taken to insure that the spillway capacity will be sufficient to (1) evacuate surcharge so that the dam will not be overtopped by a recurrent storm, and (2) prevent the surcharge from being kept partially full by a prolonged runoff whose peak, although less than the inflow design flood, exceeds the spillway capacity. To meet these requirements, the minimum spillway capacity should be in accord with the following general criteria:

(1) In the case of snow-fed perennial streams, the spillway capacity should never be less than the peak discharge of record that has resulted from snowmelt runoff. (This value may have to be estimated from a study of records on the stream itself or nearby streams.) (Secs. 34 and 41.)

(2) The spillway capacity should provide for evacuation of sufficient surcharge storage space so that in routing a succeeding "flood for assumption B," the maximum water surface does not exceed that obtained by routing the inflow design flood. In general, the recurrent storm is assumed to begin 4 days after the time of peak outflow obtained in routing the inflow design flood.

(3) In regions having an annual rainfall of 40 inches or more, the time interval to the beginning of the recurrent storm in criterion (2) should be reduced to 2 days.

(4) In regions having an annual rainfall of 20 inches or less, the time interval to the beginning of the recurrent storm in criterion (2) may be increased to 7 days.

182. ***Flood Routing.***—The accumulation of storage in a reservoir depends on the difference between the rates of inflow and outflow. For an interval of time Δt, this relationship can be expressed by the equation:

$$\Delta S = Q_i \Delta t - Q_o \Delta t \tag{1}$$

where:

ΔS=storage accumulated during Δt,

Q_i=average rate of inflow during Δt, and

Q_o=average rate of outflow during Δt.

The rate of inflow versus time curve is represented by the inflow design flood hydrograph; the rate of outflow is represented by the spillway discharge versus reservoir-elevation curve; and storage is shown by the reservoir storage versus reservoir-elevation curve. For routing studies, the inflow design flood hydrograph is not variable once selection of the inflow design flood has been made. The reservoir storage capacity also is not variable for a given reservoir site, so far as routing studies are concerned. The spillway discharge curve is variable: It depends not only on the size and type of spillway but also on the manner of operating the spillway (and outlets in some instances) to regulate the outflow.

The quantity of water a spillway can discharge depends on the type of the control device. For a simple overflow crest the flow will vary with the head on the crest, and surcharge will increase with an increase in spillway discharge. For a gated spillway, however, outflow can be varied with respect to reservoir head by operation of the

gates. For example, one assumption for an operation of a gate-controlled spillway might be that the gates will be regulated so that inflow and outflow are equal until the gates are wide open; or an assumption can be made to open the gates at a slower rate so that surcharge storage will accumulate before the gates open wide.

Outflows need not necessarily be limited to discharges through the spillway but might be supplemented by releases through the outlets. In all such cases the size, type, and method of operation of the spillway and outlets with reference to the storages or to the inflow must be predetermined in order to establish an outflow-elevation relationship.

If equations could be established for the inflow design flood hydrograph curve, the spillway discharge curve (as may be modified by operational procedures), and the reservoir storage curve, a solution of flood routing could be made by mathematical integration. However, simple equations cannot be written for the flood hydrograph curve and the reservoir storage curve, and such a solution is not practical. Many techniques of flood routing have been devised, each with its advantages and disadvantages. These techniques vary from a strictly arithmetical integration method to an entirely graphical solution. Mechanical and electronic routing machines have been developed, and digital computers have been employed. For simplicity, the arithmetical trial-and-error tabular method is illustrated in this text.

Table 18 is an example of a flood routing for the data given in figures 171, 172, and 173. These data are necessary regardless of the method of flood routing used and they consist of the following:

(1) Inflow hydrograph (rate of inflow versus time), figure 171.

(2) Reservoir capacity (reservoir storage versus reservoir elevation), figure 172.

(3) Discharge curve (rate of outflow versus reservoir elevation), figure 173.

TABLE 18. —*Flood routing computations*

(1) Time, hours	(2) Δt, hours	(3) Average rate of inflow Q, for Δt, second-feet	(4) Average inflow, acre-feet	(5) Trial reservoir storage elevation end of Δt	(6) Average rate of outflow Q_o, second-feet	(7) Average outflow for Δt, acre-feet	(8) Incremental storage ΔS, acre-feet	(9) Total storage, acre-feet	(10) Reservoir elevation end of Δt	(11) Remarks
0				300.0				1,050.0		
1	1.0	600	50.0	300.2	2	0.2	49.8	1,099.8	300.3	High.
				300.3	4	3	49.7	1,099.7	300.3	OK.
2	1.0	1,400	116.7	300.8	32	2.7	114.0	1,213.7	301.0	High
				301.0	41	3.4	113.3	1,213.0	301.0	OK
3	1.0	3,000	250.0	302.3	176	14.7	235.3	1,448.3	302.1	Low
				302.1	159	13.3	236.7	1,449.7	302.1	OK
4	1.0	5,000	416.7	303.9	470	39.2	377.5	1,827.2	308.8	Low
				303.8	450	37.5	379.2	1,828.9	303.8	OK
5	1.0	5,300	441.7	305.0	860	71.7	370.0	2,198.9	305.3	High
				305.3	910	75.8	365.9	2,194.8	305.3	OK.
6	1.0	4,000	333.3	306.3	1,350	112.5	220.8	2,415.6	306.2	Low
				306.2	1,330	110.8	222.5	2,417.3	306.2	OK
7	1.0	2,800	233.3	306.7	1,610	134.2	99.1	2,516.4	306.6	Low
				306.6	1,590	132.5	100.8	2,518.1	306.6	OK
8	1.0	2,000	166.7	306.7	1,700	141.7	25.0	2,543.1	306.7	OK
9	1.0	1,300	108.3	306.5	1,680	140.0	−31.7	2,511.4	306.5	OK
11	2.0	800	133.3	306.2	1,560	260.0	−126.7	2,384.7	306.0	Low
				306.0	1,530	255.0	−121.7	2,389.7	306.0	OK

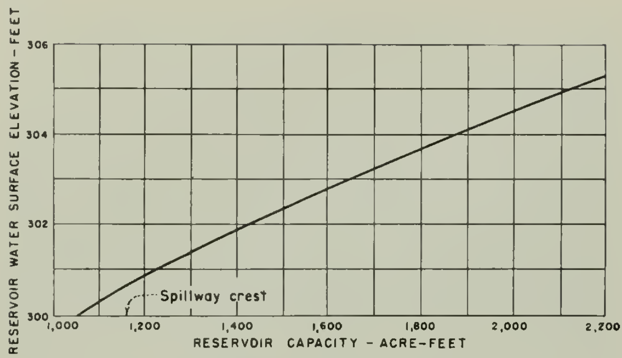


Figure 172. **Reservoir capacity curve.**

The procedure for computations shown in table 18 is as follows:

(1) Select a time interval, Δt, column (2).

(2) Obtain column (3) from the inflow hydrograph, figure 171.

(3) Obtain column (4) by converting column (3) values of second-feet for Δt to acre-feet (1 second-foot for 12 hours=1 acre-foot).

(4) Assume trial reservoir water surface in column (5) and determine the corresponding rate of outflow from figure 173.

(5) Average the rate of outflow determined in step (4) and the rate of outflow for the reservoir water surface which existed at the beginning of the period and enter in column (6).

(6) Obtain column (7) by converting column (6) values of second-feet for Δt to acre-feet, similar to step (3).

(7) Column (8) = column (4) − column (7).

(8) The initial value in column (9) represents the reservoir storage at the beginning of the inflow design flood.

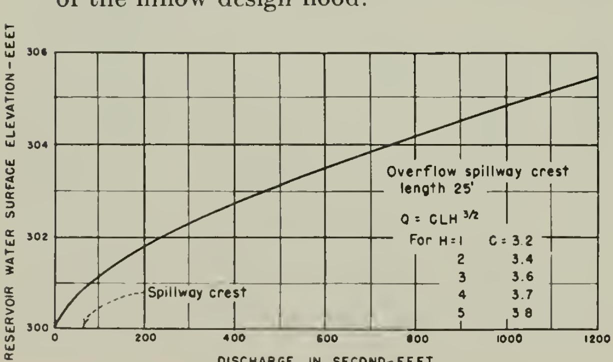


Figure 173. **Spillway discharge-elevation curve.**

Determine subsequent values by adding ΔS values from column (8) to the previous column (9) value.

(9) Determine reservoir elevation in column (10) corresponding to storage in column (9) from figure 172.

(10) Compare reservoir elevation in column (10) with trial reservoir elevation in column (5). If they do not agree within 0.1 foot, make a second trial elevation and repeat procedure until agreement is reached.

The outflow time curve resulting from the flood routing shown in table 18 has been plotted as curve B on figure 171. As the area under the inflow hydrograph (curve A) indicates the volume of inflow, so will the area under the outflow hydrograph (curve B) indicate the volume of outflow. It follows then that the volume indicated by the area between the two curves will be the surcharge storage. The surcharge storage computed in table 18 can, therefore, be checked by comparing it with the measured area on the graph.

A rough approximation of the relationship of spillway size to surcharge volume can be obtained without making an actual flood routing, by arbitrarily assuming an approximate outflow-time curve and then measuring the area between it and the inflow hydrograph. For example, if the surcharge volume for the problem shown on figure 171 is sought where a 3,000-second-foot spillway would be provided, an assumed outflow curve represented by curve C can be drawn and the area between this curve and curve A can be planimetered. Curve C will reach its apex of 3,000 second-feet where it crosses curve A. The volume represented by the area between the two curves will indicate the approximate surcharge volume necessary for this capacity spillway.

183. *Selection of Spillway Size and Type.*—(a) *General Considerations.*—In determining the best combination of storage and spillway capacity to accommodate the selected inflow design flood, all pertinent factors of hydrology, hydraulics, design, cost, and damage should be considered. In this connection and when applicable, consideration should be given to such factors as (1) the characteristics of the flood hydrograph; (2) the damages which would result if such a flood occurred without the dam; (3) the damages which would result if such a flood occurred with the dam in place; (4) the damages which would occur if

the dam or spillway were breached; (5) effects of various dam and spillway combinations on the probable increase or decrease of damages above or below the dam (as indicated by reservoir backwater curves and tailwater curves); (6) relative costs of increasing the capacity of spillways; and (7) use of combined outlet facilities to serve more than one function, such as control of releases and control or passage of floods. Service outlet releases may be permitted in passing part of the inflow design flood unless such outlets are considered to be unavailable in time of flood.

The outflow characteristics of a spillway depend on the particular device selected to control the discharge. These control facilities may take the form of an overflow weir, an orifice, a tube, or a pipe. Such devices can be unregulated or they can be equipped with gates or valves to regulate the outflow.

After a spillway control of certain dimensions has been selected, the maximum spillway discharge and the maximum reservoir water level can be determined by flood routing. Other components of the spillway can then be proportioned to conform to the required capacity and to the specific site conditions, and a complete layout of the spillway can be established. Cost estimates of the spillway and dam can then be made. Estimates of various combinations of spillway capacity and dam height for an assumed spillway type, and of alternative types of spillways, will provide a basis for selection of the economical spillway type and the optimum relation of spillway capacity to height of dam. Figures 174 and 175 illustrate the results of such a study. The relationships of spillway capacities to maximum reservoir water

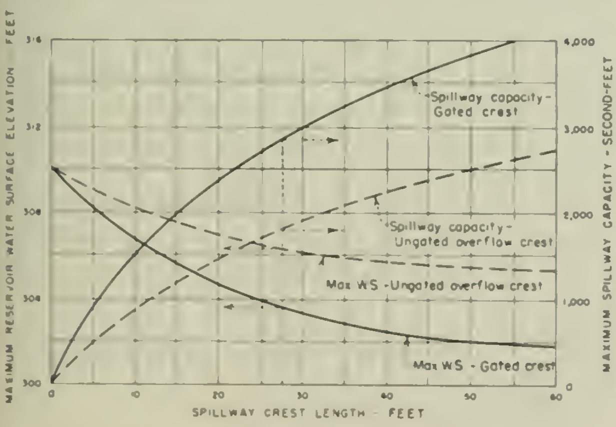


Figure 174. **Spillway capacity-surcharge relationship.**

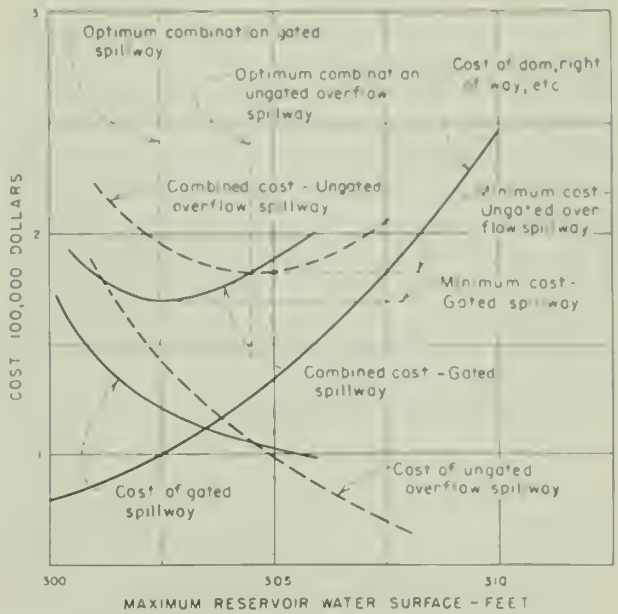


Figure 175. **Comparative costs of spillway-dam combinations.**

surfaces obtained from the flood routings is shown on figure 174 for two spillways. Figure 175 illustrates the comparative costs for different combinations of spillway and dam, and indicates a combination which results in the least total cost.

To make such a study as illustrated requires many flood routings, spillway layouts, and spillway and dam estimates. Even then, the study is not necessarily complete since many other spillway arrangements could be considered. A comprehensive study to determine alternative optimum combinations and minimum costs may not be warranted for the design of small dams. Judgment on the part of the designer would be required to select for study only the combinations which show definite advantages, either in cost or adaptability. For example, although a gated spillway might be slightly cheaper than an ungated spillway, it may be desirable to adopt the latter because of its less complicated construction, its automatic and trouble-free operation, its ability to function without an attendant, and its less costly maintenance.

(b) *Combined Service and Auxiliary Spillways.*—Where site conditions are favorable, the possibility of gaining overall economy by utilizing an auxiliary spillway in conjunction with a smaller service-type structure should be considered. In such cases the service spillway should be designed to pass floods

likely to occur frequently and the auxiliary spillway control set to operate only after such small floods are exceeded. In certain instances the outlet works may be made large enough to serve also as a service spillway. Conditions favorable for the adoption of an auxiliary spillway are the existence of a saddle or depression along the rim of the reservoir which leads into a natural waterway, or a gently sloping abutment where an excavated channel can be carried sufficiently beyond the dam to avoid the possibility of damage to the dam or other structures.

Because of the infrequency of use, it is not necessary to design the entire auxiliary spillway for the same degree of safety as required for other structures, provided the control portion of the spillway is designed to be safe, since its failure would release large flows from the reservoir. For example, concrete lining may be omitted from an auxiliary spillway channel excavated in competent rock. Where the channel is excavated through less competent material, it might be lined but terminated above the river channel with a cantilevered lip rather than extending to a stilling basin at river level. The design of auxiliary spillways is often based on the premise that some damage to portions of the structure from passage of infrequent flows is permissible. Minor damage by scour to an unlined channel, by erosion and undermining at the downstream end of the channel, and by creation of an erosion pool downstream from the spillway might be tolerated.

An auxiliary spillway can be designed with a fixed crest control, or it can be stoplogged or gated to increase the capacity without additional surcharge head. "Fuse plug" dikes which are designed to breach and wash out when overtopped often are substituted for some or all of the gates. Their advantage over gates is that, if properly designed, breaching becomes automatic whenever overtopping occurs; furthermore, they are cheaper to install and to maintain. Since the chance of their failure from overtopping is contingent on the occurrence of infrequent floods, their cost for replacement is too problematical for evaluation. By dividing the dike into short sections of varying height so that they are not all simultaneously overtopped, smaller floods might be passed with the failure of one or several of the sections, with total failure occurring only as the probable maximum flood is approached. The breaching of one section at a time will minimize the flood wave brought about by sudden failure of the dike.

Figure 176 shows the general plan and sections of the service and auxiliary spillways at Box Butte Dam. Figure 177 is an aerial photograph showing the service spillway, which consists of a "bathtub-shaped" side channel crest, a culvert conduit under the dam, a diverging concrete-lined chute, and a hydraulic-jump stilling basin. The wide auxiliary spillway channel with fuseplug control structure is shown in the top of figure 178 and the service spillway chute is in the right foreground. Note the outlet works control house and the outlet works channel which empties into the spillway stilling basin.

The Box Butte auxiliary spillway channel was excavated in a soft sandstone with only fair erosion-resistant qualities. To minimize erosion should discharge occur, the channel floor was made level so that velocities would be low. Erosion would start at the downstream end and progress slowly upstream. The control structure consists of a concrete-lined section; the cantilever lip and the downstream cutoff are provided to halt erosion upstream. The diversion walls and the fuseplug sections of varying crest elevations will insure progressive failure of the dike. The two sections nearest the dam were made the highest so that they will be the last to be overtopped. This was done to keep the flows away from the dam and to make the channel flow distance longer for discharges less than the maximum for which the spillway was designed.

(c) *Emergency Spillways.*—As the name implies, emergency spillways are provided for additional safety should emergencies not contemplated by normal design assumptions arise. Such situations could be the result of an enforced shutdown of the outlet works, a malfunctioning of spillway gates, or the necessity for bypassing the regular spillway because of damage or failure of some part of that structure. An emergency might arise where flood inflows are handled principally by surcharge storage and a recurring flood develops before a previous flood is evacuated by the small service spillway or the outlet works. Emergency spillways would act as auxiliary spillways if a flood greater than the selected inflow design flood occurred.

Under normal reservoir operation, emergency spillways are never required to function. The

control crest is, therefore, placed at or above the designed maximum reservoir water surface. The freeboard requirement for the dam is based on a water surface determined by assuming an arbitrary discharge which might result from a possible emergency. Usually, an encroachment on the freeboard provided for the designed maximum water surface is allowed in considering the design of an emergency spillway.

Emergency spillways are provided primarily to avoid an overtopping of the main dam embankment because of an emergency condition. Therefore, to be effective the emergency spillway must offer resistance to erosion greater than does the dam itself. Emergency spillways are often formed by lowering the crest of a dike section below that of the main embankment, by utilizing saddles or depressions along the reservoir rim, or by excavating channels through ridges or abutments. The exit channel of an emergency spillway should be a sufficient distance from the dam to preclude damage to the embankment or appurtenances should the spillway operate.

Figure 154 shows an emergency spillway at Wasco Dam. This spillway was provided to prevent overtopping of the embankment should the combination outlet works-spillway fail to function properly.

B. DESCRIPTION OF SERVICE SPILLWAYS

184. ***Selection of Spillway Layout.***—A composite design of a spillway can be prepared by properly considering the various factors influencing the spillway size and type, and correlating alternatively selected components. Many combinations of components can be used in forming a complete spillway layout. After the hydraulic size and outflow characteristics of a spillway are determined by routing of the design flood, the general dimensions of the control can be selected. Then, a specific spillway layout can be developed by considering the topography and foundation conditions, and by fitting the control structure and the various components to the prevailing conditions.

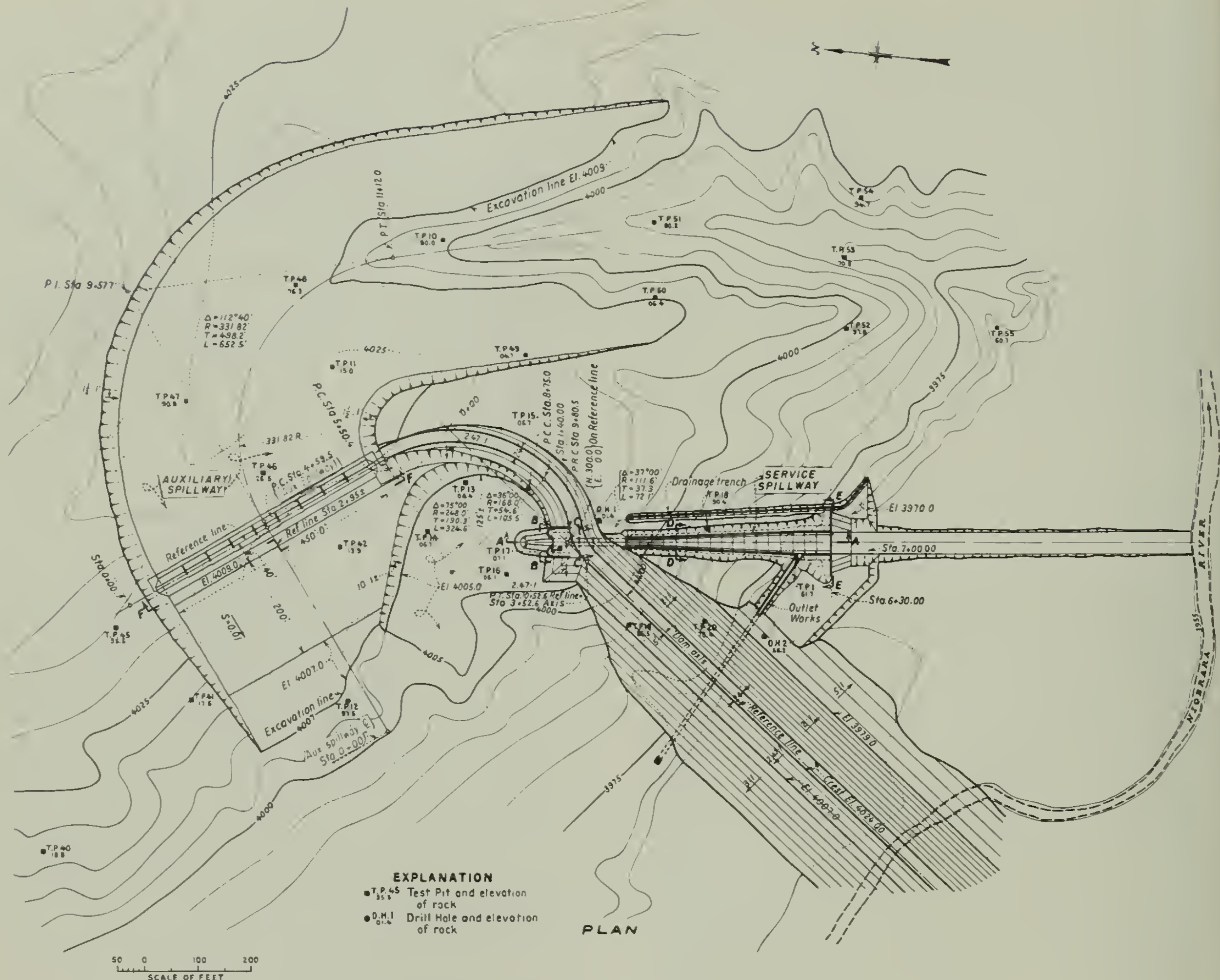
Site conditions greatly influence the selection of location, type, and components of a spillway. The steepness of the terrain traversed by the spillway control and discharge channel, the class and amount of excavation and the possibility for its use as embankment material, the chances of scour of the bounding surfaces and the need for lining, the permeability and bearing capacity of the foundation, and the stability of the excavated slopes must all be considered in the selection.

The adoption of a particular size or arrangement for one of the spillway components may influence the selection of other components. For example, a wide control structure with the crest placed normal to the centerline of the spillway would require a long, converging transition to join it to a narrow discharge channel or to a tunnel; a better alternative might be the selection of a narrower gated control structure or a side channel control arrangement. Similarly, a wide stilling basin may not be feasible for use with a cut-and-cover conduit or tunnel, because of the long, diverging transition needed.

A spillway may be an integral part of a dam such as an overflow section of a concrete dam, or it may be a separate structure. In some instances, it may be combined as a common discharge structure with the outlet works or integrated into the river diversion plan for economy. Thus, the location, type, and size of other appurtenances are factors which may influence the selection of a spillway location or its arrangement. The final plan will be governed by overall economy, hydraulic sufficiency, and structural adequacy.

The components of a spillway and common types of spillways are described and discussed herein. Hydraulic design criteria and procedures are discussed in parts C through F of this chapter.

185. ***Spillway Components.***—(a) *Control Structure.*—A major component of a spillway is the control device, since it regulates and controls the outflows from the reservoirs. This control limits or prevents outflows below fixed reservoir levels, and it also regulates releases when the reservoir rises above that level. The control structure may consist of a sill, weir, orifice, tube, or pipe. The discharge-head relationship may be fixed as in the case of a simple overflow crest or unregulated port, or it may be variable as with a gated



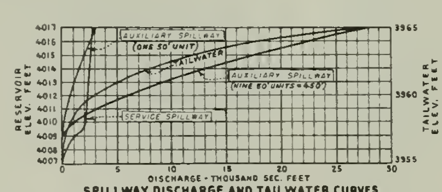


Figure 176. Service and auxiliary spillways for Box Butte Dam—Plan and sections. From drawing 278–D–49. (Sheet 1 of 2.)

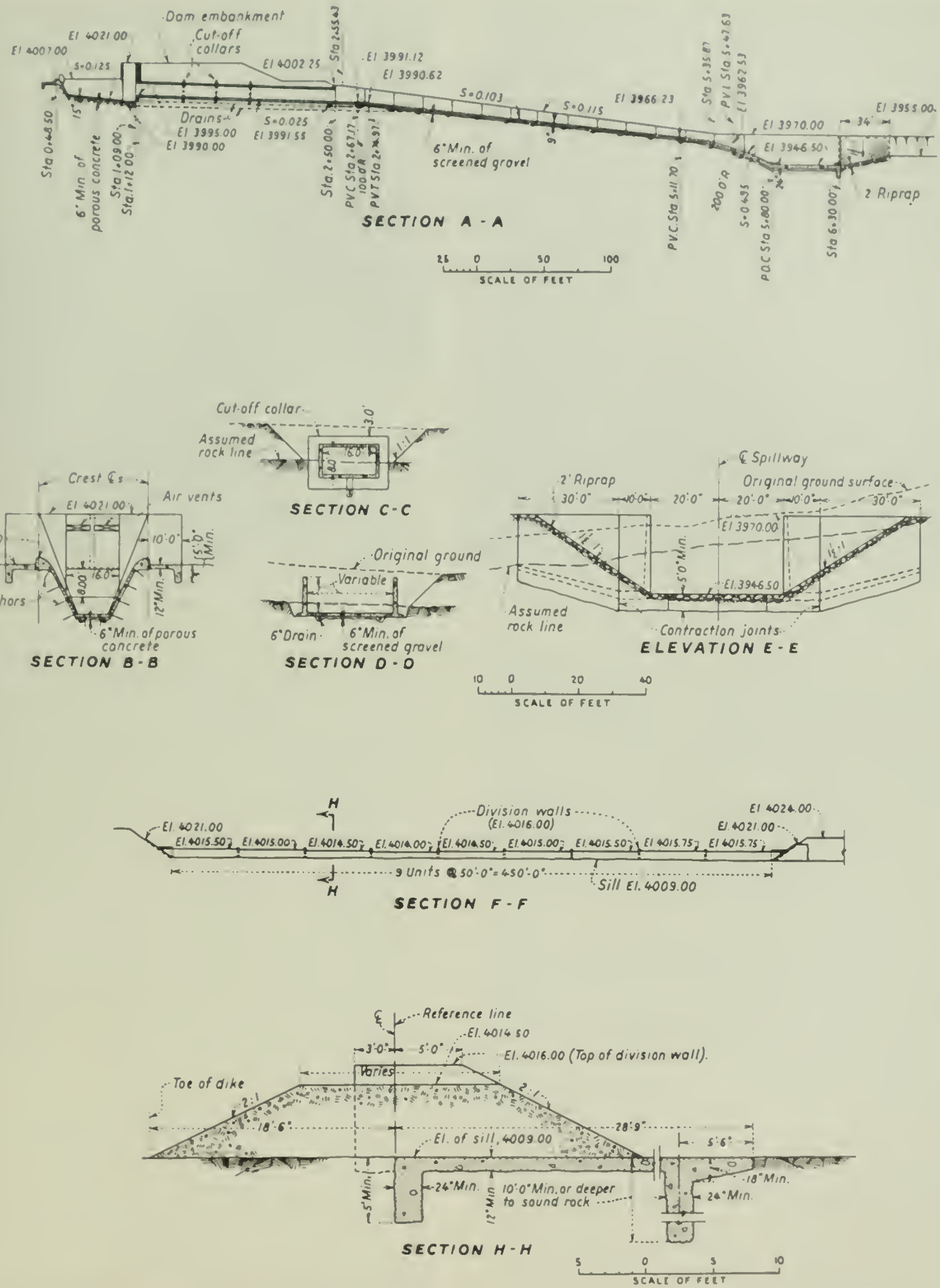


Figure 176. Service and auxiliary spillways for Box Butte Dam—Plan and sections. From drawing 278–D–49. (Sheet 2 of 2.)

Figure 177. Service spillway for Box Butte Dam.

Figure 178. Service and auxiliary spillways for Box Butte Dam.

crest or a valve-controlled pipe. The control characteristics of a closed conduit might change with the stage relationship. In a culvert spillway, for example, the entrance will act as a weir for low heads when it is not submerged and as an orifice when submerged. As the amount of submergence increases, the flow will be controlled by the conduit acting as a tube, and finally, for greater submergence, the conduit will flow full and the flow will be governed by pressure pipe characteristics.

Control structures may take various forms in both positioning and shape. In plan, overflow crests can be straight, curved, semicircular, U-shaped, or round. A semicircular crest for a small spillway is shown in figure 179. Orifice controls can be placed in a horizontal, inclined, or vertical position. The orifice can be circular, square, rectangular, triangular, or varied in shape. Tubes can be placed vertically, horizontally, or inclined; and they can be circular, square, rectangular, or varied in shape. Pipes can be straight or curved, follow any profile, and be circular, square, rectangular, horseshoe, or of other cross section.

An overflow can be sharp crested, ogee shaped, broad crested, or of varied cross section. Orifices can be sharp edged, round edged, or bellmouth shaped, and can be placed so as to discharge with a fully contracted jet or with a suppressed jet. They may discharge freely or discharge partly or fully submerged. Tubes may have entrance corners which are sharp edged, rounded, or bellmouthed; and they can be of uniform size or be divergent or convergent. Tubes can operate freely discharging or they can be partly or fully submerged. Pipes can be of uniform or changing size, with the control placed either at the downstream end or at some intermediate point along the length. Pipes can flow full under pressure for their entire length or they can flow full and partly full, respectively, above and below their control point.

(b) *Discharge Channel.*—Flow released through the control structure usually is conveyed to the streambed below the dam in a discharge channel or waterway. Exceptions are where the discharge falls free from an arch dam crest or where the flow is released directly along the abutment hillside to cascade down the abutment face. The conveyance structure may be the downstream face of a concrete dam, an open channel excavated along

Figure 179. **Semicircular overflow crest for small chute spillway at Fruitgrowers Dam in Colorado.**

the ground surface, a closed cut-and-cover conduit placed through or under a dam, or a tunnel excavated through an abutment. The profile may be variably flat or steep; the cross section may be variably rectangular, trapezoidal, circular, or of other shape; and the discharge channel may be wide or narrow, long or short.

Discharge channel dimensions are governed primarily by hydraulic requirements, but the selection of profile, cross-sectional shapes, widths, length, etc., is influenced by the geologic and topographic characteristics of the site. Open channels excavated in the abutment usually follow the ground surface profile; steep canyon walls may make a tunnel desirable. In plan, open channels may be straight or curved, with sides parallel, convergent, divergent, or a combination of these. A closed conduit may consist of a vertical or an inclined shaft leading to a nearly horizontal tunnel through the abutment or to a cut-and-cover conduit under or through the dam. Occasionally a combination of a closed conduit and an open channel might be adopted, such as a culvert under an embankment emptying into an open channel leading down the abutment slope. Discharge channels must be cut through or lined with material which is resistant to the scouring action of the accelerating velocities, and which is structurally adequate to withstand the forces from backfill, uplift, waterloads, etc.

(c) *Terminal Structure.*—When spillway flows fall from reservoir pool level to downstream river level, the static head is converted to kinetic energy. This energy manifests itself in the form of high velocities which if impeded result in large pressures. Means of returning the flow to the river without serious scour or erosion of the toe of the dam or damage to adjacent structures must usually be provided.

In some cases the discharge may be delivered at high velocities directly to the stream where the energy is absorbed along the streambed by impact, turbulence, and friction. Such an arrangement is satisfactory where erosion-resistant bedrock exists at shallow depths in the channel and along the abutments or where the spillway outlet is sufficiently removed from the dam or other appurtenances to avoid damage by scour, undermining, or abutment sloughing. The discharge channel may be terminated well above the streambed level or it may be continued to or below streambed.

Upturned deflectors, cantilevered extensions, or flip buckets can be provided to project the jet some distance downstream from the end of the structure. Often, erosion in the streambed at the point of contact of the jet can be minimized by fanning the jet into a thin sheet by the use of a flaring deflector.

Where severe scour at the point of jet impingement is anticipated, a plunge basin can be excavated in the river channel and the sides and bottom lined with riprap or concrete. No definite design criteria except that indicated in section 230 have as yet been established for determining the size or dimensions of a plunge basin which might be necessary to absorb the impact of the flow properly or to avoid scouring velocities. For small installations, it may be expedient to perform a minimum of excavation and to permit the flow to erode a natural pool; protective riprapping or concrete lining may be later provided to halt the scour. In such arrangements an adequate cutoff or other protection must be provided at the end of the spillway structure to prevent it from being undermined.

Where serious erosion to the streambed is to be avoided, the high energy of the flow must be dissipated before the discharge is returned to the stream channel. This can be accomplished by the use of an energy dissipating device, such as a hydraulic jump basin, a roller bucket, a sill block apron, a basin incorporating impact baffles and walls, or some similar energy absorber or dis-

sipator. A description of these devices and a discussion of their hydraulic design is given in part C of this chapter.

(d) *Entrance and Outlet Channels.*—Entrance channels serve to draw water from the reservoir and convey it to the control structure. Where a spillway draws water immediately from the reservoir and delivers it directly back into the river, as in the case with an overflow spillway over a concrete dam, entrance and outlet channels are not required. However, in the case of spillways placed through abutments or through saddles or ridges, channels leading to the spillway control and away from the spillway terminal structure may be required.

Entrance velocities should be limited and channel curvatures and transitions should be made gradual, in order to minimize head loss through the channel (which has the effect of reducing the spillway discharge) and to obtain uniformity of flow over the spillway crest. Effects of an uneven distribution of flow in the entrance channel might persist through the spillway structure to the extent that undesirable erosion could result in the downstream river channel. Nonuniformity of head on the crest may also result in a reduction in the discharge.

The approach velocity and depth below crest level have important influence on the discharge over an overflow crest. As is shown in section 190(a), a greater approach depth with the accompanying reduction in approach velocity will result in a larger discharge coefficient. Thus, for a given head over the crest, a deeper approach will permit a shorter crest length for a given discharge. Within the limits required to secure satisfactory flow conditions and nonscouring velocities, the determination of the relationship of entrance channel depth to channel width is a matter of economics.

Outlet channels convey the spillway flow from the terminal structure to the river channel below the dam. In some instances only a pilot channel is provided, on the assumption that scouring action will enlarge the channel during major spills. Where the channel is in a relatively nonerodible material, it should be excavated to an adequate size to pass the anticipated flow without forming a control which will affect the tailwater stage in the stilling device.

The outlet channel dimensions and its need for protection by lining or riprap will depend on the influences of scour on the tailwater. Although stilling devices are provided, it may be impossible to reduce resultant velocities below the natural velocity in the original stream, and some scouring of the riverbed, therefore, cannot be avoided. Further, under natural conditions the beds of many streams are scoured during the rising stage of a flood and filled during the falling stage by deposition of material carried by the flow. After creation of a reservoir the spillway will normally discharge clear water and the material scoured by the high velocities will not be replaced by deposition. Consequently, there will be a gradual retrogression of the downstream riverbed, which will lower the tailwater stage-discharge relationship. Conversely, scouring where only a pilot channel is provided may build up bars and islands downstream, thereby effecting an aggradation of the downstream river channel which will raise the tailwater elevation with respect to discharges. The dimensions and erosion-protective measures at the outlet channel may be influenced by these considerations.

186. *Spillway Types.*—(a) *General.*—Spillways are ordinarily classified according to their most prominent feature, either as it pertains to the control, to the discharge channel, or to some other component. Spillways often are referred to as controlled or uncontrolled, depending on whether they are gated or ungated. Commonly referred to types are the free overfall (straight drop), ogee (overflow), side channel, open channel (trough or chute), conduit, tunnel, drop inlet (shaft or morning glory), culvert, and siphon.

(b) *Free Overfall (Straight Drop) Spillways.*—A free overfall or straight drop spillway is one in which the flow drops freely from the crest. This type is suited to a thin arch or deck overflow dam or to a crest which has a nearly vertical downstream face. Flows may be free discharging, as will be the case with a sharp-crested weir control, or they may be supported along a narrow section of the crest. Occasionally the crest is extended in the form of an overhanging lip to direct small discharges away from the face of the overfall section. In free overfall spillways the underside of the nappe is ventilated sufficiently to prevent a pulsating, fluctuating jet.

Where no artificial protection is provided at the base of the overfall, scour will occur in most

streambeds and will form a deep plunge pool. The volume and depth of the hole are related to the range of discharges, the height of the drop, and the depth of tailwater. The erosion-resistant properties of the streambed material including bedrock have little influence on the size of the hole, the only effect being the time necessary to scour the hole to its full depth. Probable depths of scour are discussed in section 203. Where erosion cannot be tolerated, an artificial pool can be created by constructing an auxiliary dam downstream from the main structure, or by excavating a basin which is then provided with a concrete apron or bucket.

If tailwater depths are sufficient, a hydraulic jump will form when a free overfall jet falls upon a flat apron. It has been demonstrated that the momentum equation for the hydraulic jump may be applied to the flow conditions at the base of the fall to determine the elements of the jump.

A free overfall spillway which will be effective over a wide range of tailwater depths can be designed for use with low earthfill dams [1, 2, 3, 4].[2] An artist's conception of such a structure is shown in figure 180. It consists principally of a straight breast wall weir set at the upper end of a rectangular flume section, with its horizontal apron placed at or below streambed level. Floor blocks and an end sill are provided in this case to help in the establishment of the jump and to reduce the downstream scour. This type of structure is not adaptable for high drops on yielding foundations,

[2] Numbers in brackets refer to the bibliography, sec. 212.

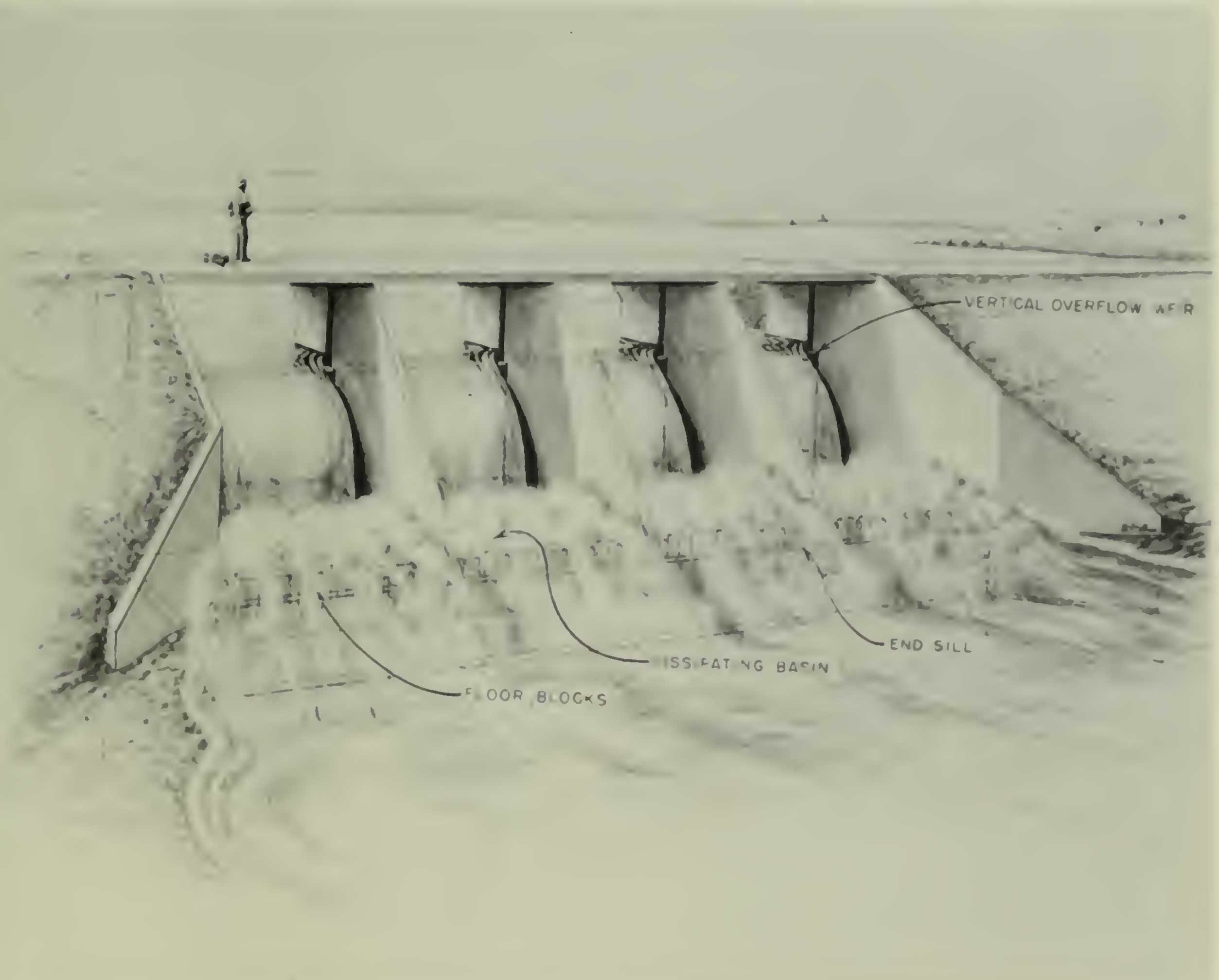


Figure 180. Typical straight drop spillway installation for small heads.

because of the large impact forces which must be absorbed by the apron at the point of impingement of the jet. Vibrations incident to the impact might crack or displace the structure, with danger from failure by piping or undermining. Ordinarily, the use of this structure for hydraulic drops from head pool to tailwater in excess of 20 feet should not be considered. The hydraulic design of the free overfall spillway is discussed in section 204.

(c) *Ogee* (*Overflow*) *Spillways*.—The ogee spillway has a control weir which is ogee or S-shaped in profile. The upper curve of the ogee ordinarily is made to conform closely to the profile of the lower nappe of a ventilated sheet falling from a sharp-crested weir. Flow over the crest is made to adhere to the face of the profile by preventing access of air to the under side of the sheet. For discharges at designed head, the flow glides over the crest with no interference from the boundary surface and attains near-maximum discharge efficiency. The profile below the upper curve of the ogee is continued tangent along a slope to support the sheet on the face of the overflow. A reverse curve at the bottom of the slope turns the flow onto the apron of a stilling basin or into the spillway discharge channel.

The upper curve at the crest may be made either broader or sharper than the nappe profile. A broader shape will support the sheet and positive hydrostatic pressure will occur along the contact surface. The supported sheet thus creates a backwater effect and reduces the efficiency of discharge. For a sharper shape, the sheet tends to pull away from the crest and to produce subatmospheric pressure along the contact surface. This negative pressure effect increases the effective head, and thereby increases the discharge.

An ogee crest and apron may comprise an entire spillway, such as the overflow portion of a concrete gravity dam, or the ogee crest may only be the control structure for some other type of spillway. Because of its high discharge efficiency, the nappe-shaped profile is used for most spillway control crests. Crest shapes and discharge coefficients are discussed in sections 188 through 190.

(d) *Side Channel Spillways*.—The side channel spillway is one in which the control weir is placed along the side of and approximately parallel to the upper portion of the spillway discharge channel. Flow over the crest falls into a narrow trough opposite the weir, turns an approximate right angle, and then continues into the main discharge channel. The side channel design is concerned only with the hydraulic action in the upstream reach of the discharge channel and is more or less independent of the details selected for the other spillway components. Flows from the side channel can be directed into an open discharge channel or into a closed conduit or inclined tunnel. Flow into the side channel might enter on only one side of the trough in the case of a steep hillside location, or on both sides and over the end of the trough if it is located on a knoll or gently sloping abutment. The "bathtub" type of side channel spillway, shown on figures 176 and 177, illustrates the latter type. Figure 181 is an artist's conception of a side channel spillway where flow enters only one side of the trough.

Discharge characteristics of a side channel spillway are similar to those of an ordinary overflow and are dependent on the selected profile of the weir crest. However, for maximum discharges the side channel flow may differ from that of the overflow spillway in that the flow in the trough may be restricted and may partly submerge the flow over the crest. In this case the flow characteristics will be controlled by a constriction in the channel downstream from the trough. The constriction may be a point of critical flow in the channel, an orifice control, or a conduit or tunnel flowing full.

Although the side channel is not hydraulically efficient nor inexpensive, it has advantages which make it adaptable to certain spillway layouts. Where a long overflow crest is desired in order to limit the surcharge head and the abutments are steep and precipitous, or where the control must be connected to a narrow discharge channel or tunnel, the side channel is often the best choice.

The hydraulic design of the side channel spillway is discussed in section 195.

(e) *Chute* (*Open Channel or Trough*) *Spillways*.—A spillway whose discharge is conveyed from the reservoir to the downstream river level through an open channel, placed either along a dam abutment or through a saddle, might be called a chute, open channel, or trough type spillway. These designations can apply regardless of the control device used to regulate the flow. Thus, a

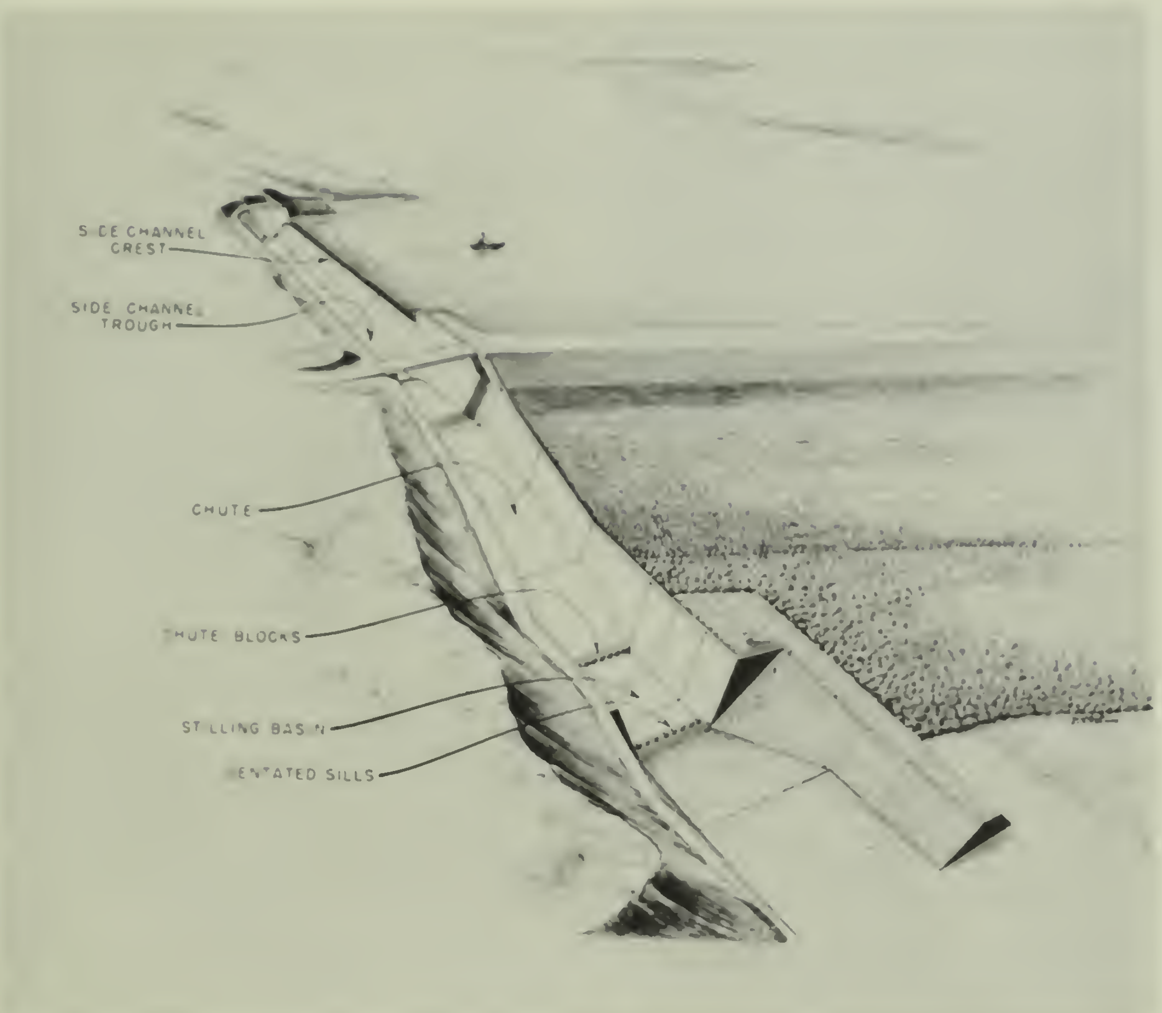


Figure 181. **Typical side channel and chute spillway arrangement.**

spillway having a chute-type discharge channel, though controlled by an overflow crest, a gated orifice, a side channel crest, or some other control device, might still be called a chute spillway. However, the name is most often applied when the spillway control is placed normal or nearly normal to the axis of an open channel, and where the streamlines of flow both above and below the control crest follow in the direction of the axis.

The chute spillway has been used with earthfill dams more often than has any other type. Factors influencing the selection of chute spillways are the simplicity of their design and construction, their adaptability to almost any foundation condition, and the overall economy often obtained by the use of large amounts of spillway excavation in the dam embankment. Chute spillways have been constructed successfully on all types of foundation materials, ranging from solid rock to soft clay.

Chute spillways ordinarily consist of an entrance channel, a control structure, a discharge channel, a terminal structure, and an outlet channel. The simplest form of chute spillway has a straight centerline and is of uniform width such as that shown in figure 182. Often, either the axis of the entrance channel or that of the discharge channel must be curved to fit the alinement to the topography. In such cases, the curvature is confined to the entrance channel if possible, because of the low approach velocities. Where the discharge channel must be curved, its floor is

Figure 182. **Chute spillway for Scofield Dam in Utah.**

sometimes superelevated to guide the high-velocity flow around the bend, thus avoiding a piling up of flow toward the outside of the chute.

Chute spillway profiles are usually influenced by the site topography and by subsurface foundation conditions. The control structure is generally placed in line with or upstream from the centerline of the dam. Usually the upper portion of the discharge channel is carried at minimum grade until it "daylights" along the downstream hillside to minimize excavation. The steep portion of the discharge channel then follows the slope of the abutment.

Flows upstream from the crest are generally at subcritical velocity, with critical velocity occurring when the water passes over the control. Flows in the chute are ordinarily maintained at supercritical stage, either at constant or accelerating rates, until the terminal structure is reached. For good hydraulic performance, abrupt vertical changes or sharp convex or concave vertical curves in the chute profile should be avoided. Similarly, the convergence or divergence in plan should be gradual in order to avoid cross waves, "ride-up" on the walls, excessive turbulence, or uneven distribution of flow at the terminal structure.

The hydraulic design of the chute spillway crest is discussed in part C, determination of hydraulic properties for the discharge channel is given in part D, and stilling basin designs are explained in part E, respectively, of this chapter.

(f) *Conduit and Tunnel Spillways.*—Where a closed channel is used to convey the discharge around or under a dam, the spillway is often called a tunnel or conduit spillway, as appropriate. The closed channel may take the form of a vertical or inclined shaft, a horizontal tunnel through earth or rock, or a conduit constructed in open cut and backfilled with earth materials. Most forms of control structures, including overflow crests, vertical or inclined orifice entrances, drop inlet entrances, and side channel crests, can be used with conduit and tunnel spillways.

With the exception of those with orifice or drop inlet entrances, tunnel and conduit spillways are designed to flow partly full throughout their length. With the drop inlet or orifice control, the tunnel or conduit size is selected so that it flows full for only a short section at the control and thence partly full for its remaining length. Ample aeration must be provided in a tunnel or conduit spillway in order to prevent a make-and-break siphonic action which would result if some part of the tunnel or conduit tends to seal temporarily because of an exhaustion of air caused by surging of the water jet, or by wave action or backwater. To guarantee free flow in the tunnel, the ratio of the flow area to the total tunnel area is often limited to about 75 percent. Air vents may be provided at critical points along the tunnel or conduit to insure an adequate air supply which will avoid unsteady flow through the spillway.

Tunnel spillways may present advantages for damsites in narrow canyons with steep abutments or at sites where there is danger to open channels from snow or rock slides. Conduit spillways may be appropriate at damsites in wide valleys, where the abutments rise gradually and are at a considerable distance from the stream channel. Use of a conduit will permit the spillway to be located under the dam near the streambed.

(g) *Drop Inlet (Shaft or Morning Glory) Spillways.*—A drop inlet or shaft spillway, as the name implies, is one in which the water enters over a horizontally positioned lip, drops through a vertical or sloping shaft, and then flows to the downstream river channel through a horizontal or near horizontal conduit or tunnel. The structure may be considered as being made up of three elements; namely, an overflow control weir, a vertical transition, and a closed discharge channel. Where the inlet is funnel-shaped, this type of structure is often called a "morning glory" or "glory hole" spillway.

Discharge characteristics of the drop inlet spill-

way may vary with the range of head. The control will shift according to the relative discharge capacities of the weir, the transition, and the conduit or tunnel. For example, as the heads increase on a glory hole spillway, the control will shift from weir flow over the crest to tube flow in the transition and then to full pipe flow in the downstream portion. Full pipe flow design for spillways except those with extremely low drops is not recommended, as is discussed in section 205(e).

A drop inlet spillway can be used advantageously at dam sites in narrow canyons where the abutments rise steeply or where a diversion tunnel or conduit is available for use as the downstream leg. Another advantage of this type of spillway is that near maximum capacity is attained at relatively low heads; this characteristic makes the spillway ideal for use where the maximum spillway outflow is to be limited. This characteristic also may be considered disadvantageous, in that there is little increase in capacity beyond the designed heads, should a flood larger than the selected inflow design flood occur. This would not be a disadvantage if this type of spillway were used as a service spillway in conjunction with an auxiliary or emergency spillway.

An artist's conception of a drop inlet spillway used with a small earthfill dam is shown in figure 183. Figure 184 shows such a conduit under construction. The hydraulic design is discussed in section 205. Additional information on the design and performance of drop inlet spillways is given in the references listed in the bibliography [5, 6, 23].

(*h*) *Culvert Spillways.* A culvert spillway is a special adaptation of the conduit or tunnel spillway. It is distinguished from the drop inlet and

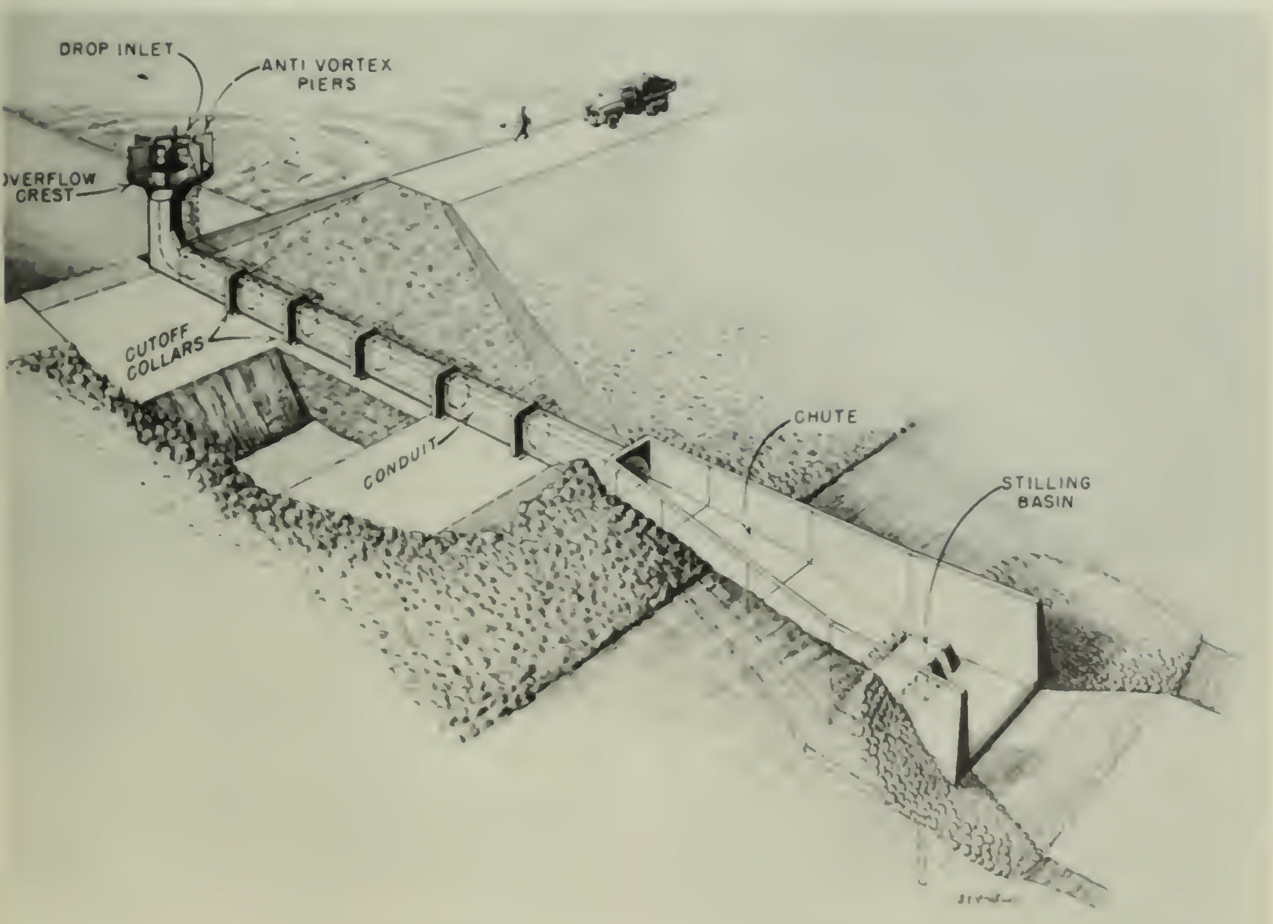


Figure 183. Drop inlet spillway for a small dam.

Figure 184. **Conduit and stilling basin for combined drop inlet spillway and outlet works at Heart Butte Dam during construction. Heart Butte Dam is on the Heart River in North Dakota.**

other conduit types in that its inlet opening is placed either vertically or inclined upstream or downstream, and its profile grade is made uniform or near uniform and of any slope. The spillway inlet opening might be sharp edged or rounded, and the approach to the conduit might have flared or tapered sidewalls with a level or sloping floor. If it is desired that the conduit flow partly full for all conditions of discharge, special precautions are taken to prevent the conduit from flowing full; if full flow is desired, bellmouth or streamlined inlet shapes are provided. Special hooded inlets are sometimes added to facilitate the flow passing from part full to full flow conditions as well as to prevent the formation of vortices which would interfere with the full flow action [24].

Culvert spillways operating with the inlet unsubmerged will act similarly to an open channel spillway. Those operating with the inlet submerged, but with the inlet orifice arranged so that full conduit flow is prevented, will act similarly to an orifice-controlled drop inlet spillway, or to

an orifice-controlled chute spillway. Where priming action is induced and the conduit flows full, the operation will be similar to that of a siphon spillway. When the culvert spillway is arranged to operate as a siphon, recognition must be taken of the disadvantages of siphon flow, especially those listed as items (4), (5), and (6) in section 186(i).

When culvert spillways placed on steep slopes flow full, reduced or negative pressures prevail along the boundaries of the conduit. Where negative pressures are large, there is danger of cavitation to the surfaces of the conduit or of its collapsing. Where cracks or joints occur along the low-pressure regions, there is the possibility of drawing in soil surrounding the conduit. Culvert spillways, therefore, should not be used for high-head installations where large negative pressures can develop. Further, the transition flow phenomenon, when the flow changes from part-full to full stage, is attended by rather severe pulsations and vibrations which increase in magnitude with increased fall of the culvert. For these reasons, culvert spillways should not be used for hydraulic drops exceeding 25 feet.

For drops not exceeding 25 feet, culvert spillways offer advantages over similar types because of their adaptability for either part-full or full flow operation and because of their simplicity and economy of construction. They might be placed on a bench excavated along the abutment on a relatively steep sidehill location, or they can be placed through the main section of the dam to discharge directly into the downstream river channel. As is the case with a drop inlet or siphon spillway, a principal disadvantage of the culvert spillway is that because its capacity does not substantially increase with increase in head, it does not provide a factor of safety against underestimation of the design flood. This disadvantage would not apply if the culvert type were used as a service spillway in conjunction with an auxiliary or emergency spillway.

The hydraulic design and details for the culvert spillway are discussed in section 206.

(i) *Siphon Spillways.*—A siphon spillway is a closed conduit system formed in the shape of an inverted U, positioned so that the inside of the bend of the upper passageway is at normal reservoir storage level. The initial discharges of the spillway, as the reservoir level rises above normal, are similar to flow over a weir. Siphonic action takes place after the air in the bend over the crest has been exhausted. Continuous flow is maintained by the suction effect due to the gravity pull of the water in the lower leg of the siphon.

Most siphon spillways are composed of five component parts, as shown on figure 185. These include an inlet, an upper leg, a throat or control section, a lower leg, and an outlet. A siphon-breaker air vent is also provided to control the siphonic action of the spillway so that it will cease operation when the reservoir water surface is drawn down to normal level. Otherwise the siphon would continue to operate until air entered the inlet. The inlet is generally placed well below the normal reservoir water surface to prevent entrance of ice and drift and to avoid the formation of vortices and drawdowns which might break the siphon action. The upper leg is formed as a bending convergent transition to join the inlet to a vertical throat section. The throat or control section is generally rectangular in cross section and is located at the crest of the upper bend of the siphon. The upper bend then continues to join a vertical or inclined tube which forms the lower leg of the siphon. Often the lower leg is placed on an adverse slope, as shown in figure 185(A), to provide a more positive priming action by forming a flow curtain which seals across the leg. The lower leg can be terminated so as to discharge vertically or along the face of a concrete dam, as shown in (B) and (C), respectively, of figure 185; or it may be provided with a lower bend and diverging outlet tube to release the flow in a horizontal direction, as shown in figure 185(A). The outlet flow can be free discharging or submerged, depending on the arrangement of the lower leg and on tailwater conditions.

A relatively simple siphon spillway which might be used with a small earthfill dam is shown on figure 186. Because of the negative pressures prevalent in the siphon, the pipe should be sufficiently rigid to withstand the collapsing forces. Joints must be made watertight, and measures must be taken to avoid cracking of the pipe from movement or settlement of the embankment. In order to prevent absolute pressures within the conduit from approaching cavitation or collapsing pressures, the total drop of the siphon should be limited to a maximum of 20 feet.

The principal advantage of a siphon spillway is its ability to pass full-capacity discharges with

narrow limits of headwater rise. A further advantage is its positive and automatic operation without mechanical devices or moving parts.

In addition to its higher cost, as compared with other types, the siphon spillway has a number of disadvantages, including the following:

(1) The inability of the siphon spillway to pass ice and debris.

(2) The possibility of clogging the siphon passageways and siphon breaker vents with debris or leaves.

(3) The possibility of water freezing in the inlet legs and air vents before the reservoir rises to the crest level of the spillway, thus preventing flow through the siphon.

(4) The occurrence of sudden surges and stoppages of outflow as a result of the erratic make-and-break action of the siphon, thus

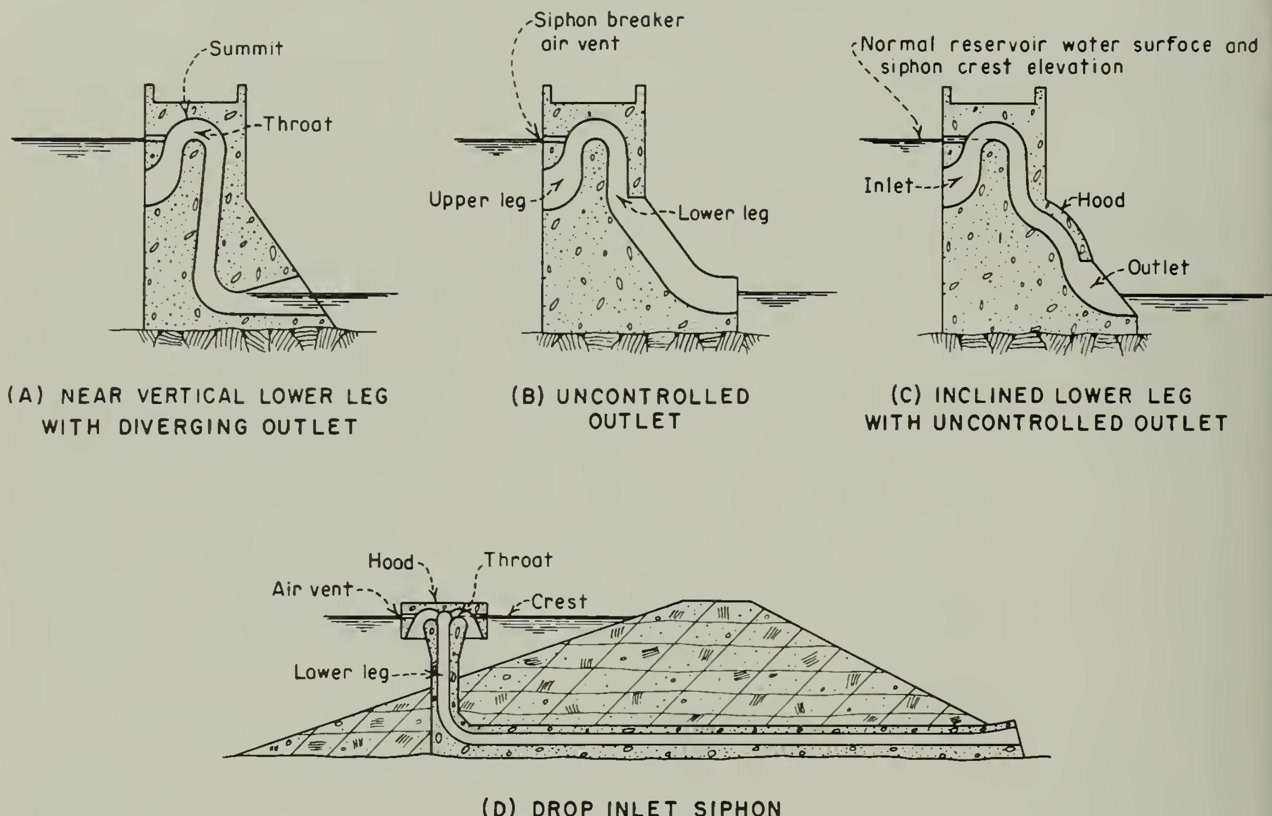


Figure 185. Typical siphon spillways.

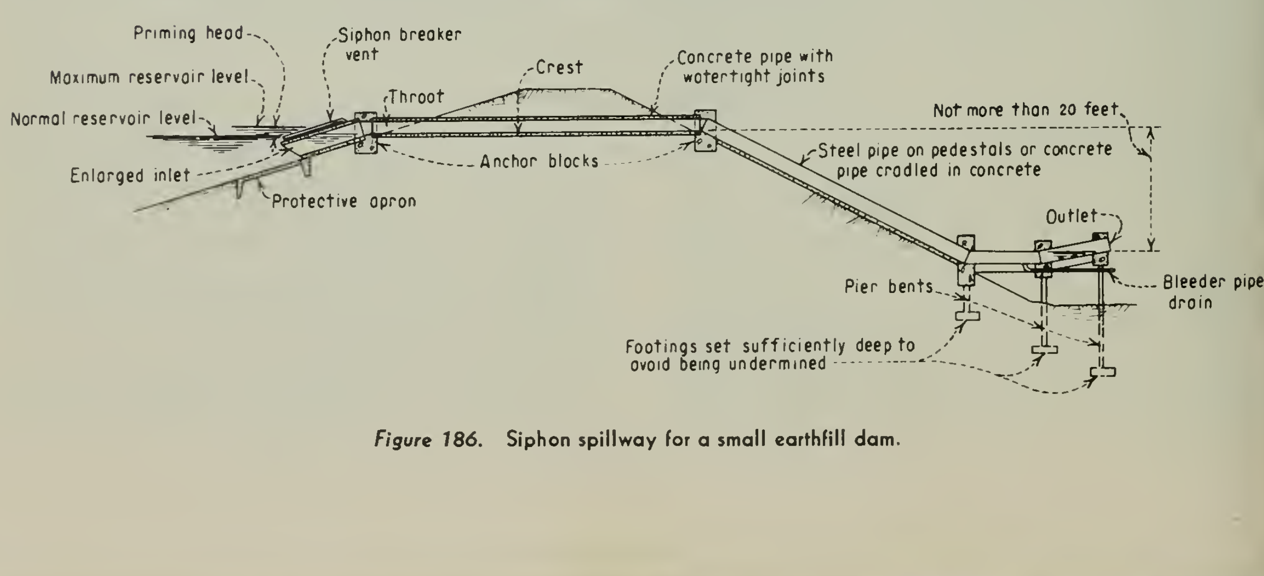


Figure 186. Siphon spillway for a small earthfill dam.

causing radical fluctuations in the downstream river stage.

(5) The release of outflows in excess of reservoir inflows whenever the siphon operates, if a single siphon is used. Closer regulation which will more nearly balance outflow and inflow can be obtained by providing a series of smaller siphons, with their siphon breaker vents set to prime at gradually increasing reservoir heads.

(6) The more substantial foundation required to resist vibration disturbances, which are more pronounced than in other types of control structures.

As is the case with other types of closed conduit structures, a principal disadvantage of the siphon spillway is its inability to handle flows materially greater than designed capacity although the reservoir head exceeds the design level. Consequently, the siphon spillway is best suited as a service spillway to be used in conjunction with an auxiliary or emergency structure.

187. *Controlled Crests.*—(a) *General.*—The simplest form of control for a spillway is the free or uncontrolled overflow crest which automatically releases water whenever the reservoir water surface rises above crest level. The advantages of the uncontrolled crest are the elimination of the need for constant attendance and regulation of the control devices by an operator, and the freedom from maintenance and repairs of the devices.

A regulating gate or other form of movable crest must be employed if a sufficiently long uncontrolled crest or a large enough surcharge head cannot be obtained for the required spillway capacity. Such devices will also be required if the spillway is to release storages below the normal reservoir water surface. The type and size of the selected control device may be influenced by such conditions as discharge characteristics of a particular device, climate, frequency and nature of floods, winter storage requirements, flood control storage and outflow provisions, the need for handling ice and debris, and special operating requirements. Whether an operator will be in attendance during periods of flood and the availability of electricity, operating mechanisms, operating bridges, etc., are factors which will influence the type of control device employed.

Many types of crest control have been devised. The type selected for a specific installation should be based on a consideration of the factors noted above as well as economy, adaptability, reliability, and efficiency. In the classification of movable crests are such devices as flashboards, stoplogs, bear-trap gates, tilting hinged-leaf gates, and drum gates. Regulating devices include stoplogs, needle beams, bulkheads, vertical or inclined rectangular lift gates, roller gates, and radial gates.

For simplicity of design and operation, only the less complicated control devices are considered appropriate for spillways for small dams. Such devices as flashboards, stoplogs, rectangular gates, and radial gates should be utilized wherever possible, since they can be easily fabricated or obtained commercially.

(b) *Flashboards and Stoplogs.*—Flashboards and stoplogs provide a means of raising the reservoir storage level above a fixed spillway crest level, when the spillway is not needed for releasing floods. Flashboards usually consist of individual boards or panels supported by vertical pins or stanchions anchored to the crest; stoplogs are boards or panels spanning horizontally between grooves recessed into supporting piers. In order to provide adequate spillway capacity, the flashboards or stoplogs must be removed before the floods occur, or they must be designed or arranged so that they can be removed while being overtopped.

Various arrangements of flashboards have been devised. Some must be placed and removed manually, some are designed to fail after being overtopped, and others are arranged to drop out of position either automatically or by being manually triggered after the reservoir exceeds a certain stage. Flashboards provide a simple economical type of movable crest device, and they have the advantage that an unobstructed crest is provided when the flashboards and their supports are removed. They have numerous disadvantages, however, which greatly limit their adaptability. Among these disadvantages are the following: (1) They present a hazard if not removed in time to pass floods, especially where the reservoir area is small and the stream is subject to flash floods; (2) they require the attendance of an operator or crew to remove them, unless they are designed to fail automatically; (3) if they are designed to fail when the water reaches certain stages their operation is uncertain, and when they fail they release sudden and undersirably large

outflows; (4) ordinarily they cannot be restored to position while flow is passing over the crest; and (5) if the spillway functions frequently the repeated replacement of flashboards may be costly.

Stoplogs are individual beams or girders set one upon the other to form a bulkhead supported in grooves at each end of the span. The spacing of the supporting piers will depend on the material from which the stoplogs are constructed, the head of water acting against the stoplogs, and the handling facilities provided for installing and removing them. Stoplogs which are removed one by one as the need for increased discharge occurs are the simplest form of a crest gate.

Stoplogs may be an economical substitute for more elaborate gates where relatively close spacing of piers is not objectionable and where removal is required only infrequently. Stoplogs which must be removed or installed in flowing water may require such elaborate hoisting mechanisms that this type of installation may prove to be as costly as gates. A stoplogged spillway requires the attendance of an operating crew for removing and installing the stoplogs. Further, the arrangement may present a hazard to the safety of the dam if the reservoir is small and the stream is subject to flash floods, since the stoplogs must be removed in time to pass the flood.

(c) *Rectangular Lift Gates.*—Rectangular lift gates span horizontally between guide grooves in supporting piers. Although these gates may be made of wood or concrete, they are often made of metal (cast iron or steel). The support guides may be placed either vertically or inclined slightly downstream. The gates are raised or lowered by an overhead hoist. Water is released by undershot orifice flow for all gate openings.

For sliding gates the vertical side members of the gate frame bear directly on the guide members; sealing is effected by the contact pressure. The size of this type of installation is limited by the relatively large hoisting capacity required to operate the gate because of the sliding friction that must be overcome.

Where larger gates are needed, wheels can be mounted along each side of the rectangular lift gates to carry the load to a vertical track on the downstream side of the pier groove. The use of wheels greatly reduces the amount of friction and thereby permits the use of a smaller hoist. Rubber or belting is used along the sides to seal the openings between the upstream leaf plate and the sides of the pier.

(d) *Radial Gates.*—Radial gates are usually constructed of steel or a combination of steel and wood. They consist of a cylindrical segment which is attached to supporting bearings by radial arms. The face segment is made concentric to the supporting pins so that the entire thrust of the waterload passes through the pins; thus, only a small moment need be overcome in raising and lowering the gate. Hoisting loads then consist of the weight of the gate, the friction between the side seals and the piers, and the frictional resistance at the pins. The gate is often counterweighted to partially counterbalance the effect of its weight, which further reduces the required capacity of the hoist.

The small hoisting effort needed to operate radial gates makes hand operation practical on small installations which otherwise might require power. The small hoisting forces involved also make the radial gate more adaptable to operation by relatively simple automatic control apparatus. Where a number of gates are used on a spillway, they might be arranged to open automatically at successively increasing reservoir levels, or only one or two might be equipped with automatic controls, while the remaining gates would be operated by hand or power hoists.

Small radial gates which may be operated either automatically or by hoist operation are available commercially. These gates are fabricated from structural steel members and have either a corrugated-metal or plate-steel faceplate.

C. HYDRAULICS OF CONTROL STRUCTURES

188. *Shape for Uncontrolled Ogee Crest.*—As discussed in section 186(c), crest shapes which approximate the profile of the under nappe of a jet flowing over a sharp-crested weir provide the ideal form for obtaining optimum discharges. The shape of such a profile depends upon the head, the inclination of the upstream face of the overflow section, and the height of the overflow section

above the floor of the entrance channel (which influences the velocity of approach to the crest). Crest shapes have been studied extensively in the Bureau of Reclamation hydraulic laboratories, and data from which profiles for overflow crests can be obtained have been published [7]. For most conditions the data can be summarized according to the form shown on figure 187(A), where the profile is defined as it relates to axes at the apex of the crest. That portion upstream from the origin is defined as either a single curve and a tangent or as a compound circular curve. The portion downstream is defined by the equation:

$$\frac{y}{H_o}=-K\left(\frac{x}{H_o}\right)^n \qquad (2)$$

in which K and n are constants whose values depend on the upstream inclination and on the velocity of approach. Figure 187 gives values of these constants for different conditions.

The approximate profile shape for a crest with a vertical upstream face and negligible velocity of approach is shown on figure 188. The profile is constructed in the form of a compounded circular curve with radii expressed in terms of the design head, H_o. This definition is simpler than that shown on figure 187, since it avoids the need for solving an exponential equation; further, it is represented in a form easily used by a layman for constructing forms or templates. For ordinary conditions of design of small spillways and where the approach height, P, is equal to or greater than one-half the maximum head on the crest, this profile is sufficiently accurate to avoid seriously reduced crest pressures and does not materially alter the hydraulic efficiency of the crest. When the approach height is less than one-half the maximum head on the crest, the profile should be determined from figure 187.

189. *Discharge Over An Uncontrolled Overflow Ogee Crest.*—(a) *General.*—The discharge over an ogee crest is given by the formula:

$$Q=CLH_e^{3/2} \qquad (3)$$

where:

Q=discharge,

C=a variable coefficient of discharge,

L=effective length of crest, and

H_e=total head on the crest, including velocity of approach head, h_a.

The discharge coefficient, C, is influenced by a number of factors, such as (1) the depth of approach, (2) relation of the actual crest shape to the ideal nappe shape, (3) upstream face slope, (4) downstream apron interference, and (5) downstream submergence. The effect of these various factors is discussed in section 190.

The total head on the crest, H_e, does not include allowances for approach channel friction losses or other losses due to curvature of the upstream channel, entrance loss into the inlet section, and inlet or transition losses. Where the design of the approach channel results in appreciable losses, they must be added to H_e to determine reservoir elevations corresponding to the discharges given by the above equation.

(b) *Pier and Abutment Effects.*—Where crest piers and abutments are shaped to cause side contractions of the overflow, the effective length, L, will be less than the net length of the crest. The effect of the end contractions may be taken into account by reducing the net crest length as follows:

$$L=L'-2(NK_p+K_a)H_e \qquad (4)$$

where:

L=effective length of crest,

L'=net length of crest,

N=number of piers,

K_p=pier contraction coefficient,

K_a=abutment contraction coefficient, and

H_e=total head on crest.

The pier contraction coefficient, K_p, is affected by the shape and location of the pier nose, the thickness of the pier, the head in relation to the design head, and the approach velocity. For conditions of design head, H_o, average pier contraction coefficients may be assumed as follows:

	K_p
For square-nosed piers with corners rounded on a radius equal to about 0.1 of the pier thickness	0.02
For round-nosed piers	0.01
For pointed-nose piers	0

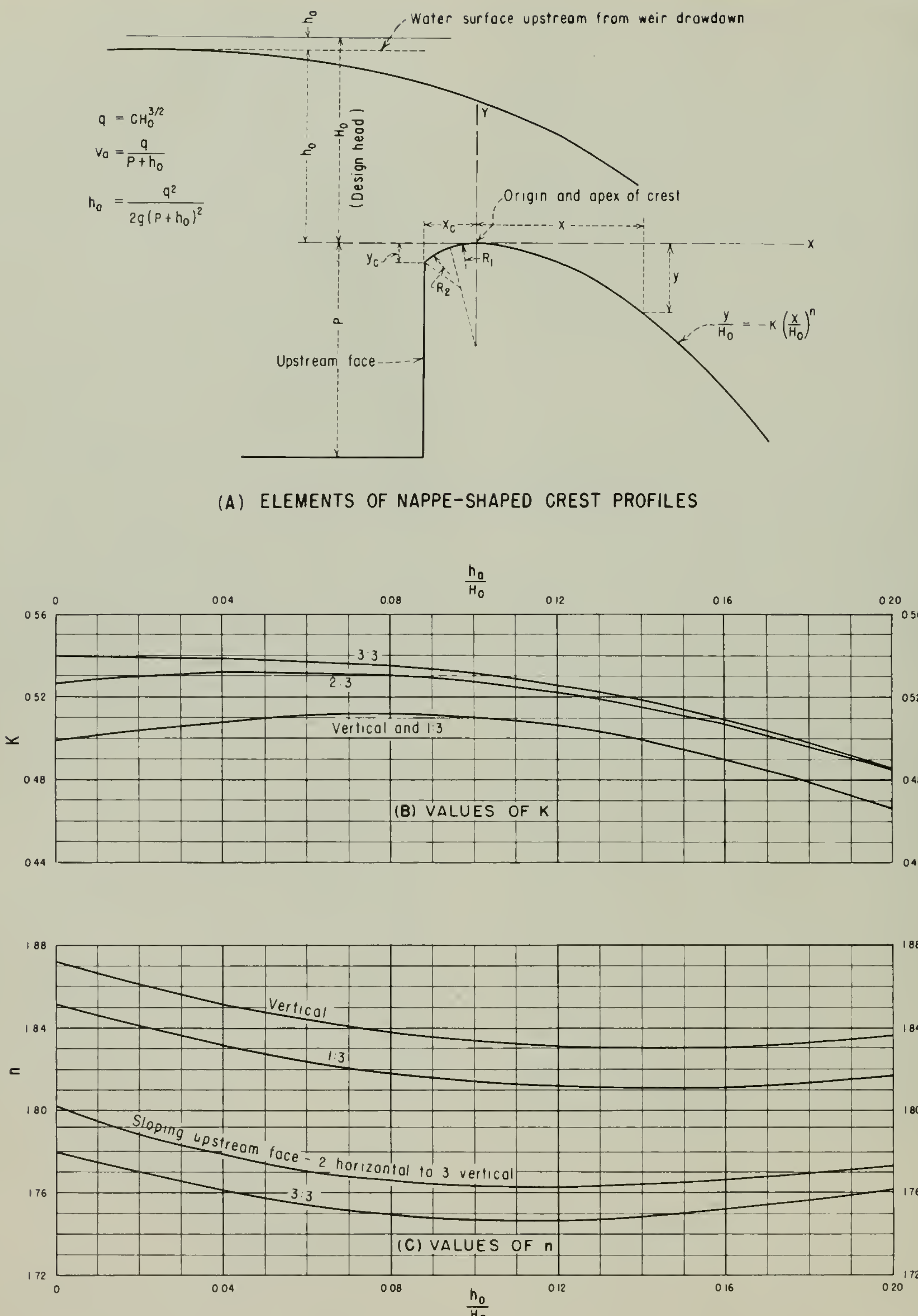


Figure 187. **Factors for definition of nappe-shaped crest profiles. (Sheet 1 of 2.)**

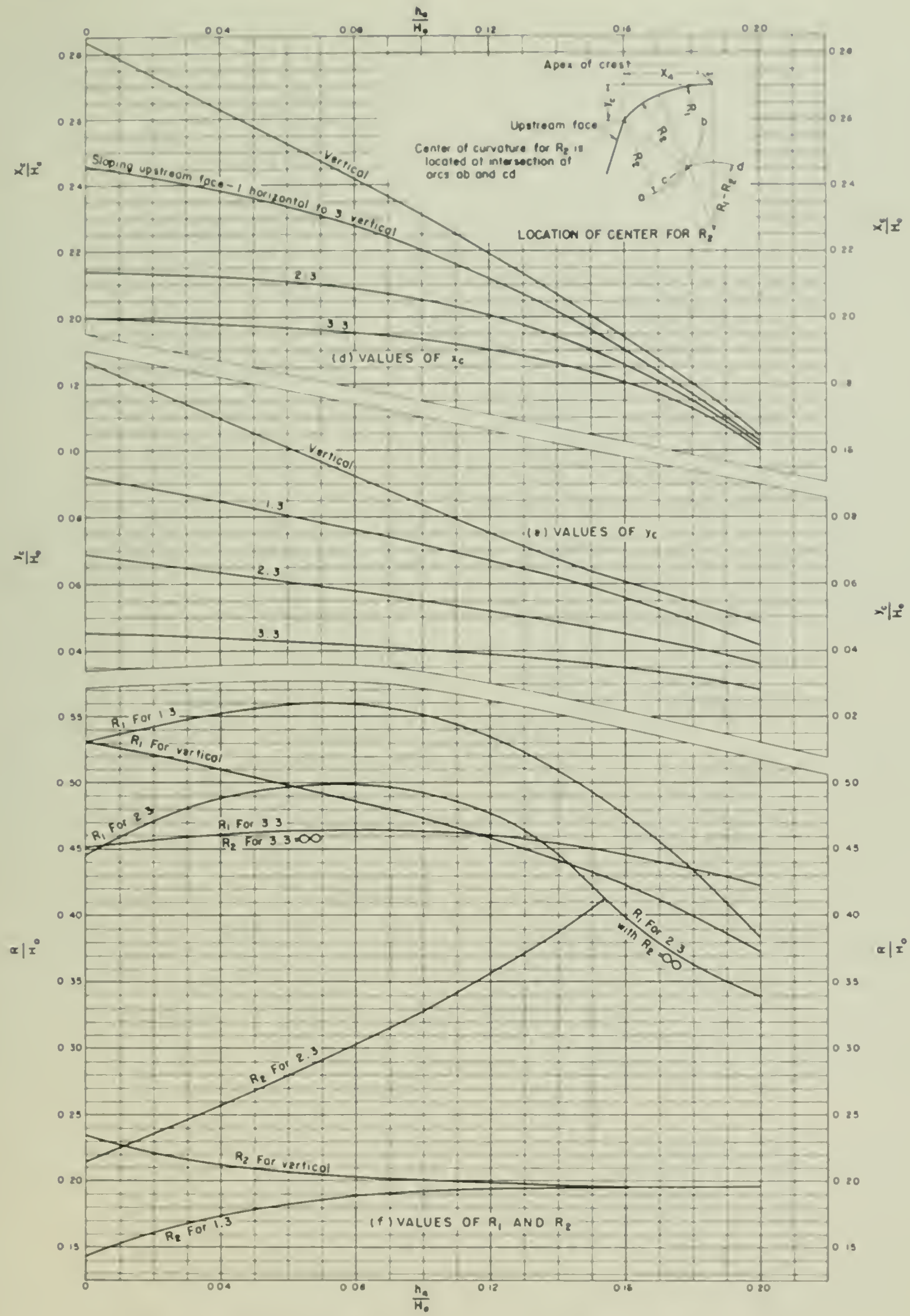


Figure 187. Factors for definition of nappe-shaped crest profiles. (Sheet 2 of 2.)

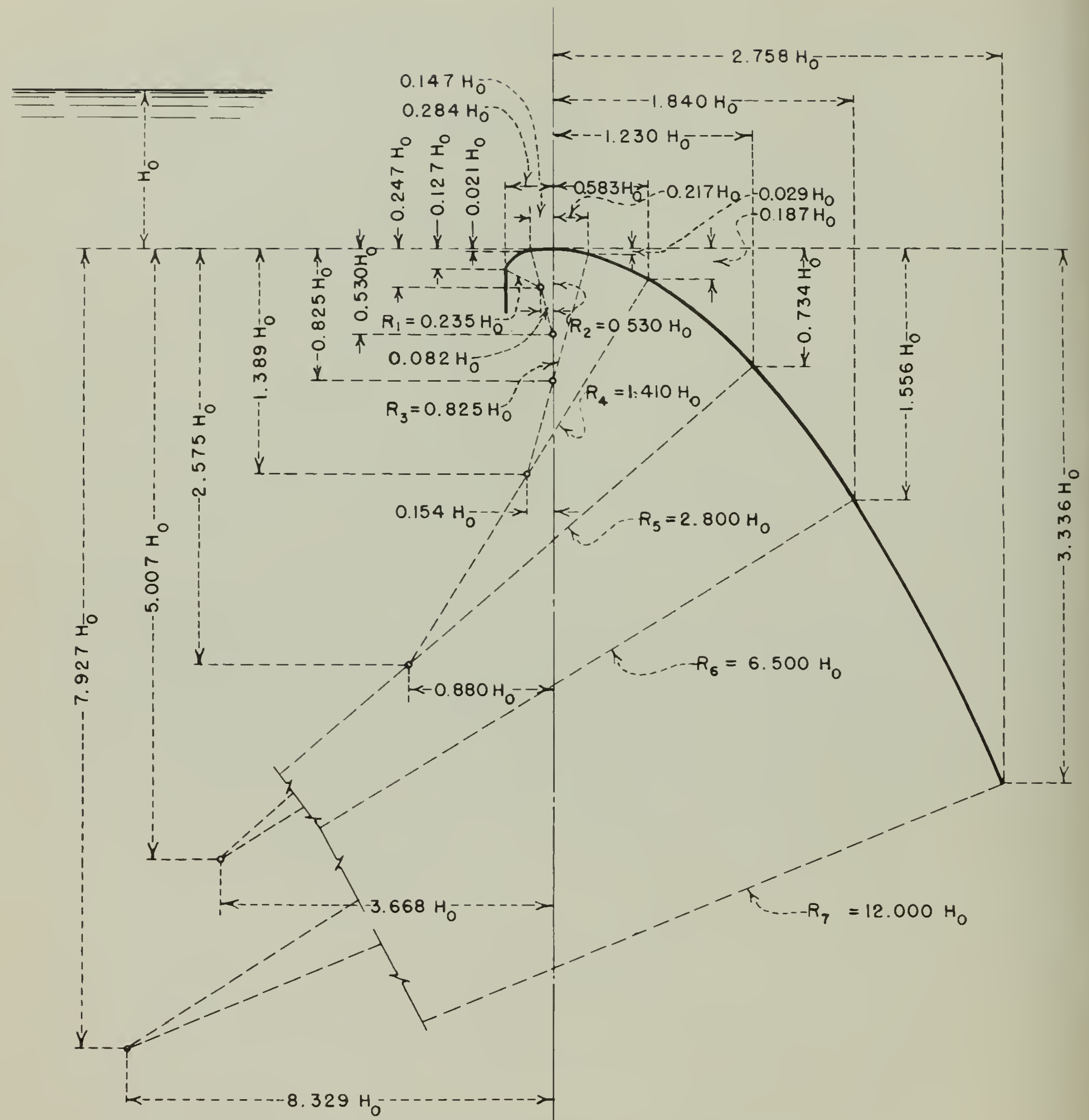


Figure 188. **Ogee crest shape defined by compound curves.**

The abutment contraction coefficient is affected by the shape of the abutment, the angle between the upstream approach wall and the axis of flow, the head in relation to the design head, and the approach velocity. For conditions of design head, H_o, average coefficients may be assumed as follows:

	K_a
For square abutments with headwall at 90° to direction of flow	0.20
For rounded abutments with headwall at 90° to direction of flow, when $0.5H_o \geqq r \geqq 0.15H_o$	0.10
For rounded abutments where $r > 0.5H_o$ and headwall is placed not more than 45° to direction of flow	0

where r = radius of abutment rounding.

190. *Coefficient of Discharge for Uncontrolled Ogee Crests.* (a) *Effect of Depth of Approach.* For a high sharp-crested weir placed in a channel, the velocity of approach is small and the under side of the nappe flowing over the weir attains maximum vertical contraction. As the approached depth is decreased, the velocity of approach increases and the vertical contraction diminishes. For sharp-crested weirs whose heights are not less than about one-fifth the heads producing flow over them, the coefficient of discharge remains fairly constant with a value of about 3.3 although the contraction diminishes. For weir heights less than about one-fifth the head, the contraction of the flow becomes increasingly suppressed and the crest coefficient decreases. When the weir height becomes zero, the contraction is entirely suppressed and the overflow weir becomes in effect a channel or a broad-crested weir, for which the theoretical coefficient of discharge is 3.087. If the sharp-crested weir coefficients are related to the head measured from the point of maximum contraction instead of to the head above the sharp crest, coefficients applicable to ogee crests shaped to profiles of under nappes for various approach velocities can be established. The relationship of the ogee crest coefficient, C_o, to various values of $\frac{P}{H_o}$ is shown on figure 189. These coefficients are valid only when the ogee is formed to the ideal nappe shape, that is when $\frac{H_e}{H_o}=1$.

(b) *Effect of Heads Differing from Design Head.* When the ogee crest is formed to a shape differing from the ideal shape or when the crest has been shaped for a head larger or smaller than the one under consideration, the coefficient of discharge will differ from that shown on figure 189. A widened shape will result in positive pressures along the crest contact surface, thereby reducing the discharge; with a narrower crest shape negative pressures along the contact surface will occur, resulting in an increased discharge. Figure 190 shows the variation of the coefficient as related to values of $\frac{H_e}{H_o}$, where H_e is the actual head being considered.

An approximate coefficient of discharge for an irregularly shaped crest whose profile has not been formed according to the under nappe of the overflow jet can be estimated by finding an ideal shape which most nearly matches it. The design head, H_o, corresponding to the matching shape can then be used as a basis for determining the coefficients.

The coefficients for partial heads on the crest, for preparing a discharge-head relationship, can be determined from figure 190.

(c) *Effect of Upstream Face Slope.*—For small ratios of the approach depth to head on the crest, sloping the upstream face of the overflow results in an increase in the coefficient of discharge. For large ratios the effect is a decrease of the coefficient. Within the range considered in this text, the coefficient of discharge is reduced for large ratios of $\frac{P}{H_o}$ only for relatively flat upstream slopes. Figure 191 shows the ratio of the coefficient for an overflow ogee crest with a sloping face to the coefficient for a crest with a vertical upstream face as obtained from figure 189 (and as adjusted by figure 190 if appropriate), as related to values of $\frac{P}{H_o}$.

(d) *Effect of Downstream Apron Interference and Downstream Submergence.*—When the water level below an overflow weir is high enough to affect the discharge, the weir is said to be submerged. The vertical distance from the crest of the overflow to the downstream apron and the depth of flow in the downstream channel, as it relates to the head pool level, are factors which alter the coefficient of discharge.

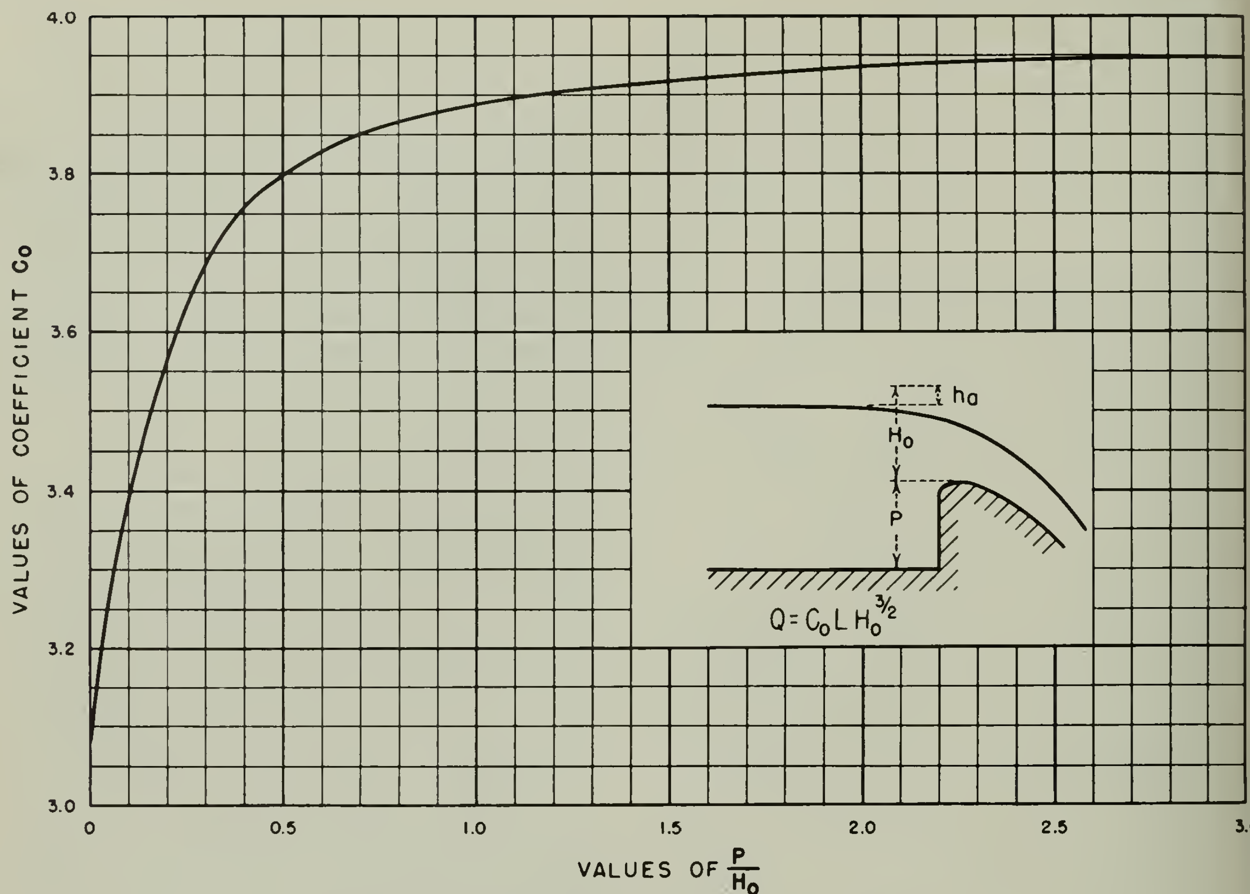


Figure 189. Discharge coefficients for vertical-faced ogee crest.

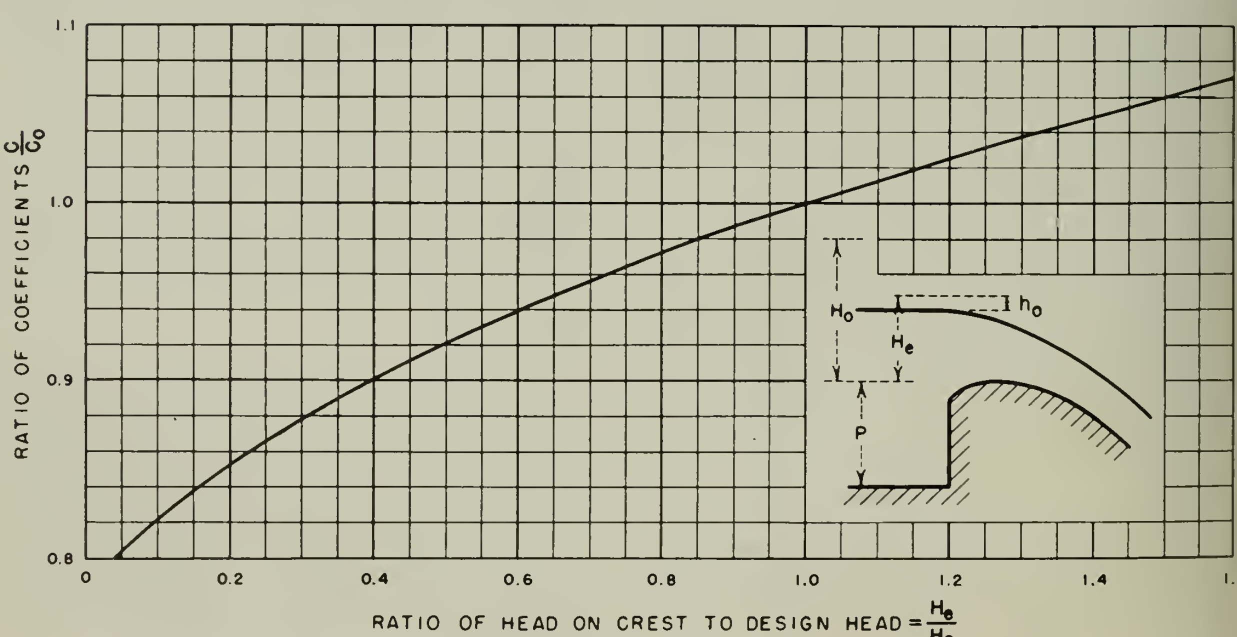


Figure 190. Coefficient of discharge for other than the design head.

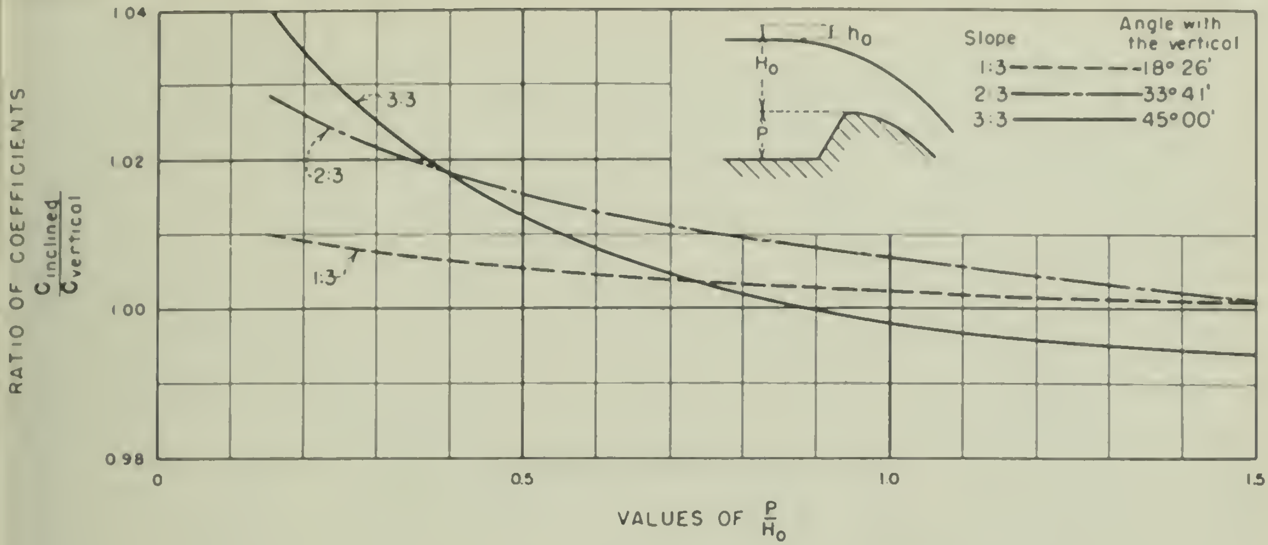


Figure 191. **Coefficient of discharge for ogee-shaped crest with sloping upstream face.**

Five distinct characteristic flows can occur below an overflow crest, depending on the relative positions of the apron and the downstream water surface: (1) Flow will continue at supercritical stage; (2) a partial or incomplete hydraulic jump will occur immediately downstream from the crest; (3) a true hydraulic jump will occur; (4) a drowned jump will occur in which the high-velocity jet will follow the face of the overflow and then continue in an erratic and fluctuating path for a considerable distance under and through the slower water; and (5) no jump will occur—the jet will break away from the face of the overflow and ride along the surface for a short distance and then erratically intermingle with the slow moving water underneath. Figure 192 shows the relationship of the floor positions and downstream submergences which produce these distinctive flows.

Where the downstream flow is at supercritical stage or where the hydraulic jump occurs, the decrease in the coefficient of discharge is due principally to the back-pressure effect of the downstream apron and is independent of any submergence effect due to tailwater. Figure 193 shows the effect of downstream apron conditions on the coefficient of discharge. It will be noted that this curve plots the same data represented by the vertical dashed lines on figure 192 in a slightly different form. As the downstream apron level nears the crest of the overflow $\left(\frac{h_d+d}{H_e}\text{ approaches }1.0\right)$, the coefficient of discharge is about 77 percent of that for unretarded flow. On the basis of a coefficient of 4.0 for unretarded flow over a high weir, the coefficient when the weir is submerged will be about 3.08, which is virtually the coefficient for a broad-crested weir.

From figure 192 it can be seen that when the $\frac{h_d+d}{H_e}$ values exceed about 1.7, the downstream floor position has little effect on the coefficient, but there is a decrease in the coefficient caused by tailwater submergence. Figure 194 shows the ratio of the coefficient of discharge where affected by tailwater conditions, to the coefficient for free flow conditions. This curve plots the data represented by the horizontal dashed lines on figure 192 in a slightly different form. Where the dashed lines on figure 192 are curved, the decrease in the coefficient is the result of a combination of tailwater effects and downstream apron position.

191. *Examples of Designs of Uncontrolled Ogee Crests.* The following two examples illustrate the methods of designing uncontrolled ogee crests, including the computation of approach channel losses and velocity head, the determination of the total length of the crest, and the correction of the coefficient of discharge for various effects.

(a) *Example 1.*—Design an uncontrolled overflow ogee crest for a chute spillway, to discharge 2,000 second-feet at a 5-foot head, and prepare a discharge-head curve. The upstream face of the crest is sloped 1:1, and the entrance channel is 100 feet long. A bridge is to span the crest, and 18-inch-wide bridge piers with rounded noses are to be provided. The bridge spans are not to

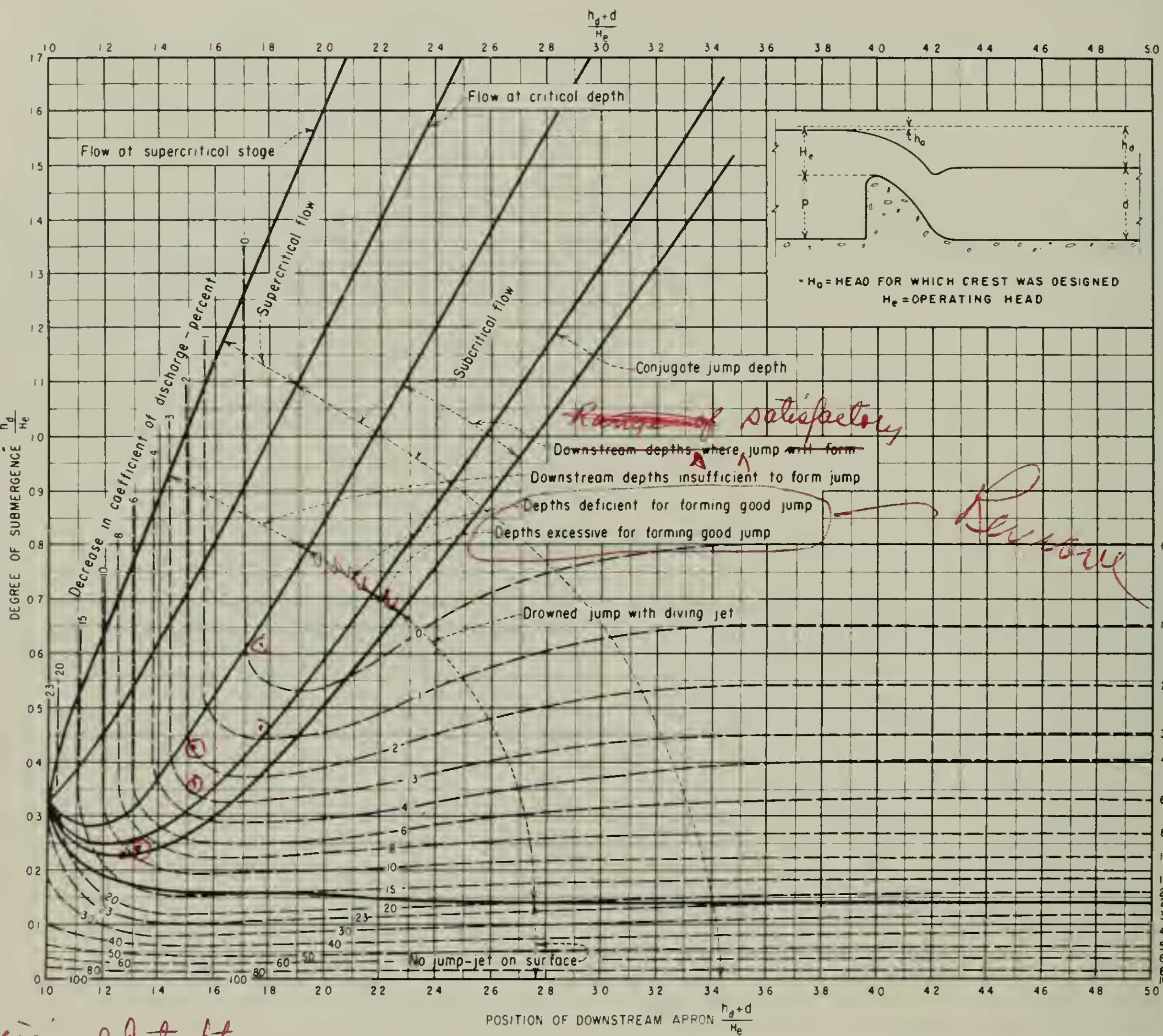


Figure 192. Effects of downstream influences on flow over weir crests.

exceed 20 feet. The abutment walls are rounded to a 5-foot radius, and the approach walls are to be placed at 30° with the centerline of the spillway entrance.

To solve the problem, either the approach depth and apron position with respect to the crest must be selected and the appropriate coefficient determined, or an arbitrary coefficient must be selected and the appropriate dimensions determined. The solution will show both procedures.

Procedure 1: First, assume the position of the approach and downstream apron levels with respect to the crest level, say 2 feet below crest level. Then $H_e + P$ is approximately 7 feet.

To evaluate the approach channel losses, assume a value of C to obtain an approximate approach velocity, say $C=3.7$. Then the discharge per unit of crest length, q, is equal to $CH_e^{3/2}=3.7\times5^{3/2}=41$ second-feet. The velocity of approach, v_a, is then equal to $\frac{q}{H_e+P}=\frac{41}{7}=5.9$ feet per second, and the approach velocity head, h_a, is equal to $\frac{v^2_a}{2g}=\frac{5.9^2}{64.4}=0.5$ feet.

Assuming a value of 0.0225 for the friction coefficient, n, in Manning's formula and assuming the hydraulic radius, r, equal to the depth of approach, the friction slope, s, is equal to

$$\left(\frac{v_a n}{1.486r^{2/3}}\right)^2=\left(\frac{5.7\times0.0225}{1.486\times7^{2/3}}\right)^2=0.0006.$$

Then the total approach channel friction loss, h_f, is equal to $100 \times 0.0006 = 0.06$ feet. Assuming

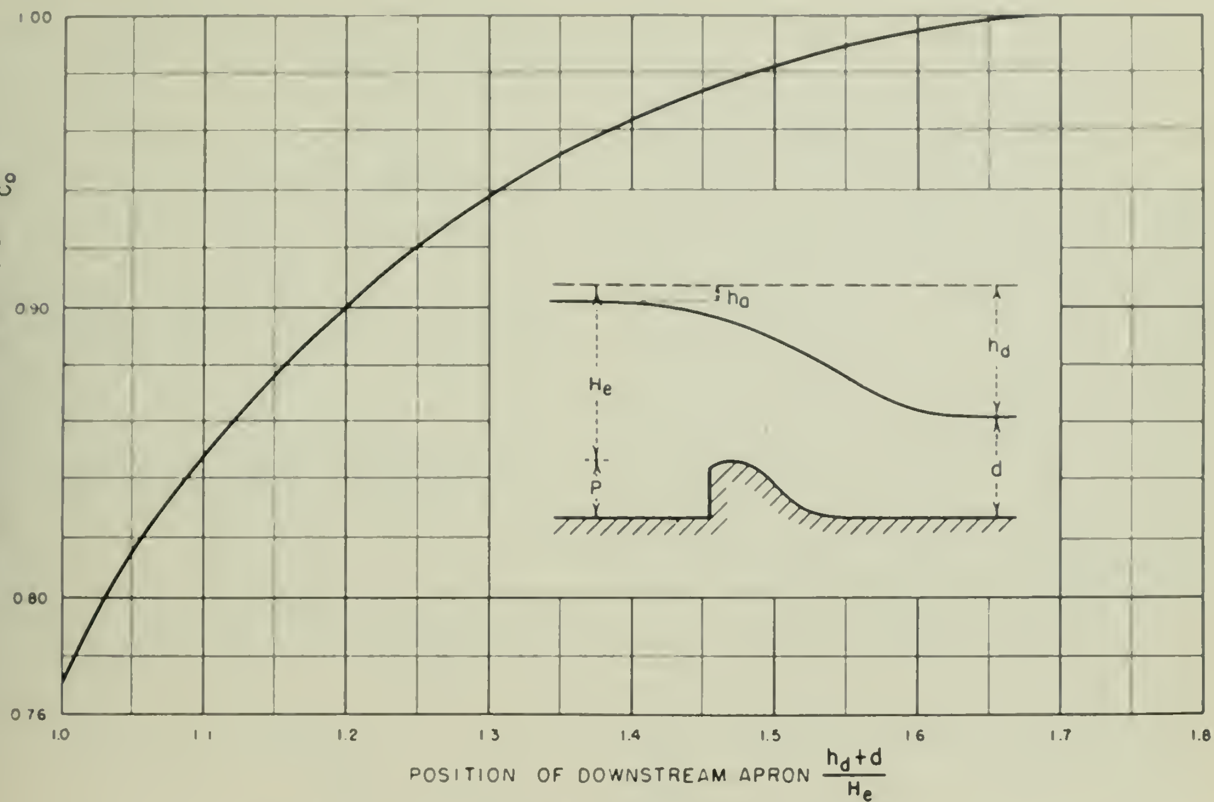


***Figure 193.* Ratio of discharge coefficients due to apron effect.**

an entrance loss into the approach channel equal to 0.1 h_a, the total loss of head in the approach is approximately 0.06 + 0.1 × 0.5 = 0.11 feet.

The effective head, H_o, is equal to 5.0−0.11 =4.89 feet, and $\frac{P}{H_o}$ is equal to $\frac{2}{4.89}=0.41$. From figure 189, the value of C_o for a $\frac{P}{H_o}$ value of 0.41 is 3.77.

Figure 191 is used to correct the discharge coefficient for the inclined upstream slope. For a 1:1 slope and a value of 0.41 for $\frac{P}{H_o}$, the ratio of $\frac{C_{inclined}}{C_{vertical}}$ is 1.018. Then, $C_i=1.018\times3.77=3.84$.

Next, the relationships of $\frac{h_d+d}{H_e}$ and $\frac{h_d}{H_e}$ are evaluated to determine the downstream effects. The value of $\frac{h_d+d}{H_e}$ is approximately $\frac{6.89}{4.89}=1.41$. From figure 192 for an $\frac{h_d+d}{H_e}$ value of 1.41, the value of $\frac{h_d}{H_e}$ at supercritical flow is 0.91. If supercritical flow prevails, h_d should be equal to 0.91 $H_e=0.91\times4.89=4.44$, and d should be equal to $6.89-4.44=2.45$ feet. With the indicated unit discharge of approximately 41 second-feet, the downstream velocity will be approximately $\frac{41}{2.45}=16.7$ feet per second, and the velocity head, h_v, will be equal to $\frac{16.7^2}{64.4}=$ 4.4 feet. The closeness of the values of h_d and h_v verifies that the flow is supercritical. From figure 192 it can be seen that the downstream effect is due to apron influences only, and that the corrections shown on figure 193 will apply. The ratio of the modified C_s to the coefficient C_o for a downstream apron position determined by the $\frac{h_d+d}{H_e}$ ratio of 1.41 is 96.6 percent. The corrected coefficient therefor is $3.84\times0.966=3.71$. This coefficient has now been corrected for all influencing effects.

The next step is to determine the required crest length. For the design head, H_o, of 4.89 feet the required effective crest length, L, is equal to $\frac{Q}{CH_o^{3/2}}=\frac{2,000}{3.71\times(4.89)^{3/2}}=49.9$ feet. To correct for pier effects, the net length from equation (4) is:

$$L'=L+2(NK_p+K_a)H.$$

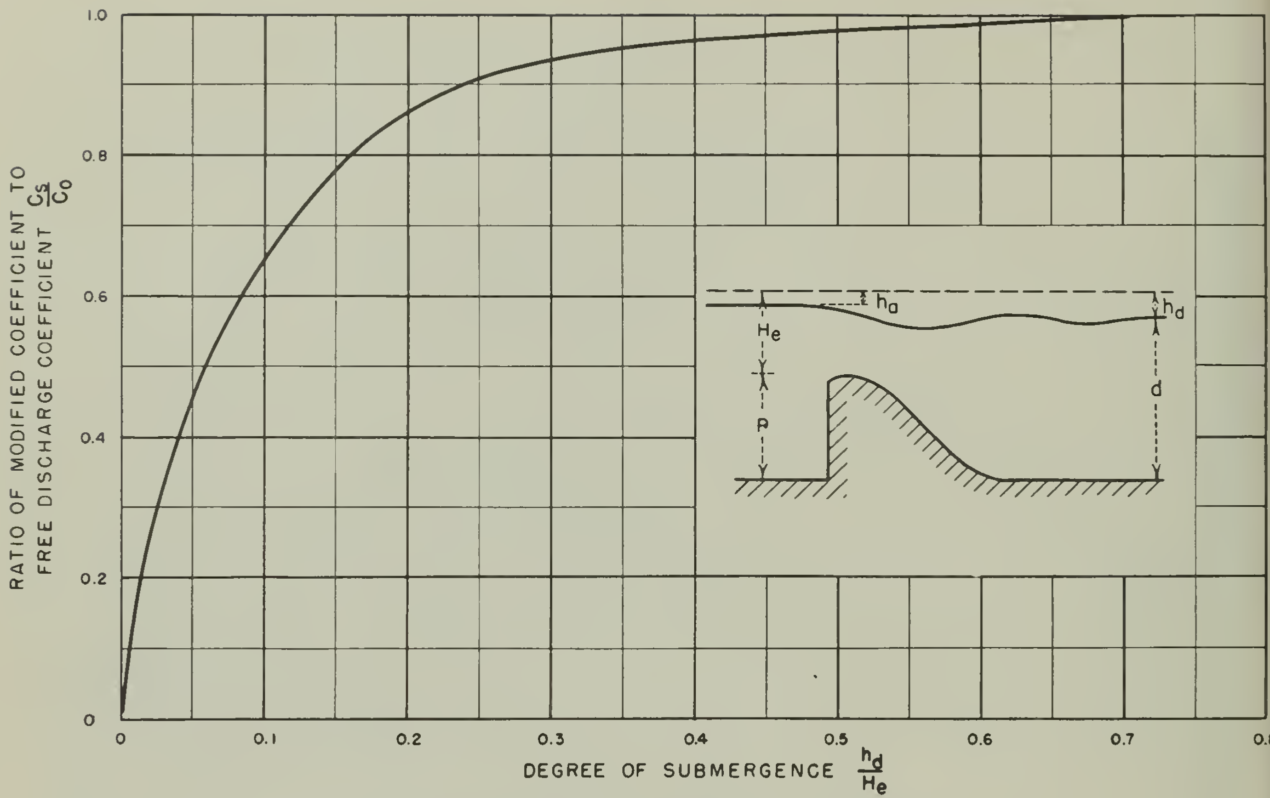


Figure 194. **Ratio of discharge coefficients due to tailwater effect.**

If the bridge spans are not to exceed 20 feet, two piers will be required for the approximate 50-foot total span and N will equal 2. Then

$$L'=49.9+2(2\times 0.01+0)4.89=50.1.$$

The foregoing procedure establishes a coefficient of discharge for the design head. For computing a rating curve, coefficients for lesser heads must be obtained. Since the variations of the different corrections are not consistent, the procedure for correcting the coefficients must be repeated for each lesser head. The variables can be tabulated in a form similar to that used in table 19.

TABLE 19.—*Design of an uncontrolled overflow ogee crest—example 1, procedure 1*

[Given l=50 feet.[1] H_o=4.89 feet. P=2 feet]

$\frac{H_e}{H_o}$	H_e feet	(2) $\frac{C}{C_o}$	C_i	h_d+d	$\frac{h_d+d}{H_e}$	(4) $\frac{C_s}{C}$	C_e	$q=C_eH_e^{3/2}$	H_e+P	v_a (approx.)	h_a	s	Entrance loss, 0.1 h_a	Total approach losses, feet	Gross head, feet	Total discharge, second-feet, $Q=C_e LH_e^{3/2}$
0.1	0.49	0.82	3.15	2.49	5.08	1.00	3.15	1.1	2.49	0.44	0.003	0.00001	0	0	0.49	55
.2	.98	.85	3.26	2.98	3.04	1.00	3.26	3.2	2.98	1.07	.02	.00006	0	.01	.99	160
.4	1.96	.90	3.46	3.96	2.02	1.00	3.46	9.5	3.96	2.40	.09	.0002	.01	.03	1.99	475
.6	2.93	.94	3.61	4.93	1.68	1.00	3.61	18.1	4.93	3.67	.21	.0004	.02	.06	2.99	905
.8	3.91	.97	3.73	5.91	1.51	.982	3.66	28.3	5.91	4.79	.36	.0005	.04	.09	4.00	1,415
1.0	4.89	1.0	[3] 3.84	6.89	1.41	.966	3.71	40.0	6.89	5.80	.52	.0006	.05	.11	5.00	2,000
1.2	5.87	1.03	3.96	7.87	1.34	.95	3.76	53.5	7.87	6.80	.72	.0007	.07	.14	6.01	2,675

[1] The effective crest length and the net crest length for H_o are 49.9 feet and 50.1 feet, respectively. Because of the small magnitude of the pier effects, an average length of 50 feet is taken for the effective crest length for all values of H_e. If the pier effects are significant, separate effective crest lengths should be computed for each H_e value.

[2] From fig. 190.

[3] $C_{inclined}$ for H_o.

[4] From fig. 193.

Procedure 2: First, assume an overall coefficient of discharge, say 3.5. The discharge per unit length, q, is then equal to $3.5H_e^{3/2}=39.2$ second-feet for $H_e=5$ feet. Then the required effective length of the crest, L, is equal to $\frac{Q}{q}=\frac{2,000}{39.2}=51$ feet.

Next, the approach depth is approximated by use of figure 189. From this figure, for $C=3.5$ the value of $\frac{P}{H_o}$ is approximately 0.2. Thus, the approach depth cannot be less than 1 foot. To allow for other factors which may reduce the coefficient, an approach depth of about 2 feet might reasonably be assumed.

With a 2-foot approach depth, the computation for approach losses will be the same as in the procedure 1 solution and the effective head, H_o, will become 4.89 feet. Similarly, the value of C_t will be 3.84.

Since the overall coefficient of 3.5 was assumed for the 5-foot gross head, the corresponding coefficient for the 4.89-foot effective head will be

$$\frac{C_o}{C_{gross\ head}}=\frac{H^{3/2}_{gross\ head}}{H^{3/2}_{effective\ head}}$$

or $C_o=C_g\left(\frac{H_g}{H_e}\right)^{3/2}=3.5\left(\frac{5.0}{4.89}\right)^{3/2}=1.035\times3.5=3.62.$

The submergence ratio, $\frac{C_s}{C_o}$, will then be $\frac{3.62}{3.84}=0.94$, and, from figure 193, $\frac{h_d+d}{H_e}$ will be 1.3. Thus h_d+d will be $1.3\times4.89=6.4$ feet. The downstream apron should therefore be placed 1.4 feet below the crest level.

Since it was demonstrated previously that pier and contraction effects are small, they can be neglected in this example, and the net crest length is, therefore, 51 feet. This crest length and downstream apron position can be varied by altering the assumptions of overall coefficient and approach depth.

The discharge rating curve may be developed by a process similar to that used in procedure 1.

(b) *Example 2.*—Design an uncontrolled overflow crest for a diversion dam to pass 2,000 second-feet with a depth of flow upstream from the dam not exceeding 5 feet above the crest. The overflow dam is 8 feet high. The abutment headwall is 90° to the direction of flow and the edge adjacent to the crest is rounded to a 12-inch radius. For 2,000 second-feet flow, the tailwater will rise 3.5 feet above the crest.

For an approximate head, H_e, of 5 feet, a crest height of 8 feet, and a crest submergence of 3.5 feet, $\frac{h_d+d}{H_e}=\frac{13}{5}=2.6$, and $\frac{h_d}{H_e}=\frac{1.5}{5}=0.3$. From figure 192 it can be seen that for these relations the downstream flow phenomena will be that of a drowned jump and that the coefficient will be reduced about 6 percent.

Roughly, $\frac{P}{H}=\frac{8}{5}=1.6$ and the unretarded coefficient from figure 189 is 3.93. Reducing this by 6 percent because of submergence results in an approximate coefficient of 3.7.

The approximate discharge per foot of crest, q, is equal to $CH_o^{3/2}=3.7\times5^{3/2}=41.5$ second-feet. Then the velocity of approach, v_a, is $\frac{41.5}{13}=3.2$ feet per second and the approach velocity head, h_a, is 0.16 feet. H_o is $5.0+0.16=5.16$ feet.

The revised value of $\frac{P}{H_o}$ does not appreciably alter the coefficient obtained from figure 189. The revised value of $\frac{h_d+d}{H_o}$ will be $\frac{13.16}{5.16}=2.55$ and the revised value of $\frac{h_d}{H_o}$ will be $\frac{1.66}{5.16}=0.32$. The reduction in coefficient due to submergence effects from figure 192 is 5 percent. The revised coefficient is 95 percent of $3.93=3.73$.

The effective crest length, L, is equal to $\frac{Q}{CH_o^{3/2}}$ $=\frac{2,000}{3.73\times(5.16)^{3/2}}=45.7$ feet.

The net crest length is determined by use of equation (4). Without piers the net crest length, L', is equal to $L+2K_aH_e$. For 90° abutment walls rounded to a radius larger than $0.15\ H_o$, $K_a=0.10$. Then the net crest length, L', is equal to $45.7+2\times0.10\times5.16=46.7$ feet.

192. *Uncontrolled Ogee Crests Designed For Less Than Maximum Head.*—Economy in the design of an ogee crest may sometimes be effected by using a design head less than the maximum expected, for determining the ogee profile. As discussed previously, use of a smaller head for design results in increased discharges for the full range of heads. The increase in capacity makes it possible to achieve economy by reducing either the crest length or the maximum surcharge head.

Tests have shown that the subatmospheric pressures on a nappe-shaped crest do not exceed about one-half the design head when the design head is not less than about 75 percent of the maximum head. For most conditions in the design of small spillways, these negative pressures will be small, and they can be tolerated because they will not approach absolute pressures which might induce cavitation. Care must be taken, however, in forming the surface of the crest where these negative pressures will occur, since unevenness caused by abrupt offsets, depressions, or projections will amplify the negative pressures to a magnitude where cavitation conditions can develop.

The negative pressure on the crest may be resolved into a system of forces acting both upward and downstream. These forces should be considered in analyzing the structural stability of the crest structure.

An approximate force diagram of the subatmospheric pressures when the design head used to determine the crest shape is 75 percent of the maximum head is shown on figure 195. These data are based on average results of tests made on ideal shaped weirs with negligible velocities of approach. Pressures for intermediate head ratios can be assumed to vary linearly, considering that no subatmospheric pressure prevails when $\frac{H_o}{H_e}$ is equal to 1.

193. *Gate-Controlled Ogee Crests.*—Releases for partial gate openings for gated crests will occur as orifice flow. With full head on the gate and with the gate opened a small amount, a free discharging trajectory will follow the path of a jet issuing from an orifice. For a vertical orifice the path of the jet can be expressed by the parabolic equation:

$$-y=\frac{x^2}{4H} \tag{5}$$

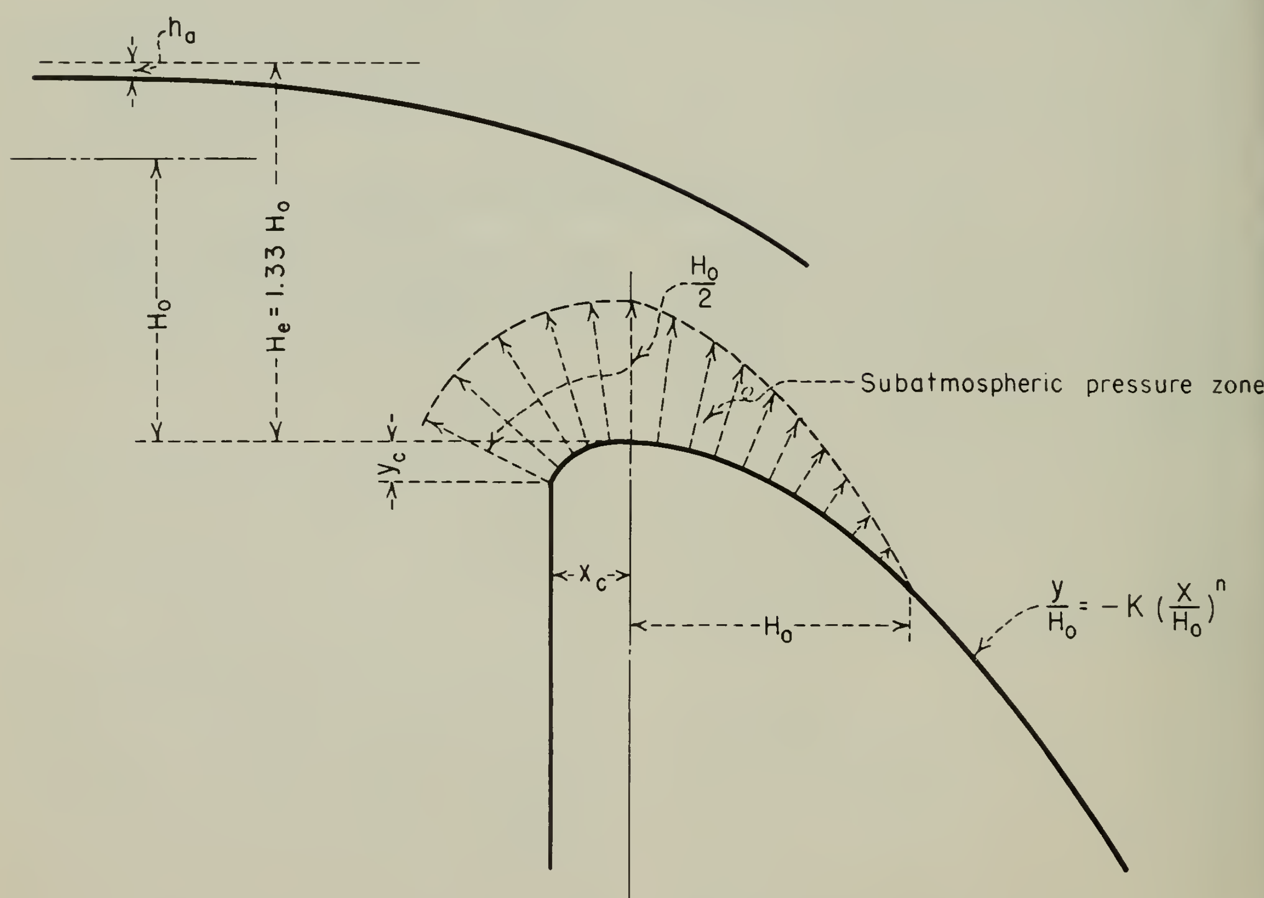


Figure 195. Subatmospheric crest pressures for $\frac{H_o}{H_e}=0.75$.

where H is the head on the center of the opening. For an orifice inclined an angle of θ from the vertical, the equation will be:

$$y = x \tan\theta + \frac{x^2}{4H\cos^2\theta} \qquad (6)$$

If subatmospheric pressures are to be avoided along the crest contact, the shape of the ogee downstream from the gate sill must conform to the trajectory profile.

Experiments have shown that when gates are operated with small openings under high heads, negative pressures will occur along the crest in the region immediately below the gate if the ogee shape is thinner than one which would conform to the trajectory shape. Tests showed the subatmospheric pressures would be equal to about one-tenth of the design head if the ogee is shaped to the ideal nappe profile for maximum head and if the gate is operated at small openings. The force diagram for this condition is shown on figure 196.

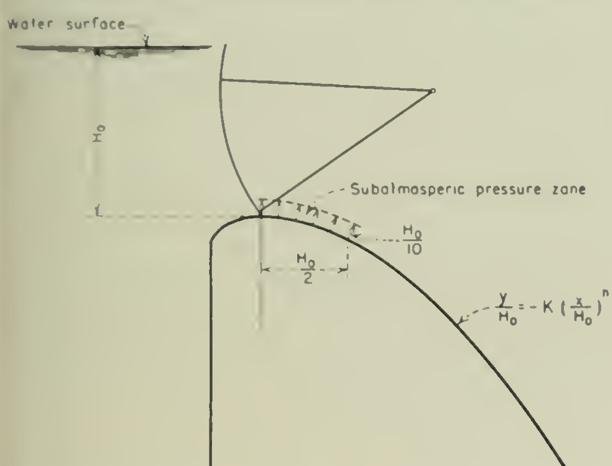


***Figure 196.* Subatmospheric crest pressures for undershot gate flow.**

The adoption of a trajectory profile rather than a nappe profile downstream from the gate sill will result in a wider ogee, and reduced discharge efficiency for full gate opening. Where the discharge efficiency is unimportant and where a wider ogee shape is needed for structural stability, the trajectory profile may be adopted to avoid subatmospheric pressure zones along the crest. Where the ogee is shaped to the ideal nappe profile for maximum head, the subatmospheric pressure area can be minimized by placing the gate sill downstream from the crest of the ogee. This will provide an orifice which is inclined downstream for small gate openings and thus will result in a steeper trajectory more nearly conforming to the nappe-shaped profile.

194. *Discharge Over Gate-Controlled Ogee Crests.* The discharge for a gated ogee crest at partial gate openings will be similar to flow through a low-head orifice and may be computed by the equation:

$$Q = \frac{2}{3}\sqrt{2g}\, CL\left(H_1^{3/2} - H_2^{3/2}\right) \qquad (7)$$

where H_1 and H_2 are the total heads (including the velocity head of approach) to the bottom and top of the orifice, respectively. The coefficient, C, will differ with different gate and crest arrangements; it is influenced by the approach and downstream conditions as they affect the jet contractions. Thus, the top contraction for a vertical leaf gate will differ from that for a curved, inclined radial gate; the upstream floor profile will affect the bottom contraction of the issuing jet; and the downstream profile will affect the back pressure and consequently the effective head. Figure 197 shows coefficients of discharge for orifice flows for various ratios of gate opening to total head. The curve represents averages determined for the various approach and downstream conditions described and is sufficiently reliable for determining discharges for small spillway structures.

195. *Side Channel Spillways.* (a) *General.*—The theory of flow in a side channel spillway [8] is based principally on the law of conservation of linear momentum, assuming that the only forces producing motion in the channel result from the fall in the water surface in the direction of the axis. This premise assumes that the entire energy of the flow over the crest is dissipated through its intermingling with the channel flow and is therefore of no assistance in moving the water along the channel. Axial velocity is produced only after the incoming water particles join the channel stream.

For any short reach of the side channel, the momentum at the beginning of the reach plus any increase in momentum due to external forces must equal the momentum at the end of the reach. If a short reach, Δx in length, is considered and the velocity and discharge at the upstream section are v and Q, respectively, at the downstream sec-

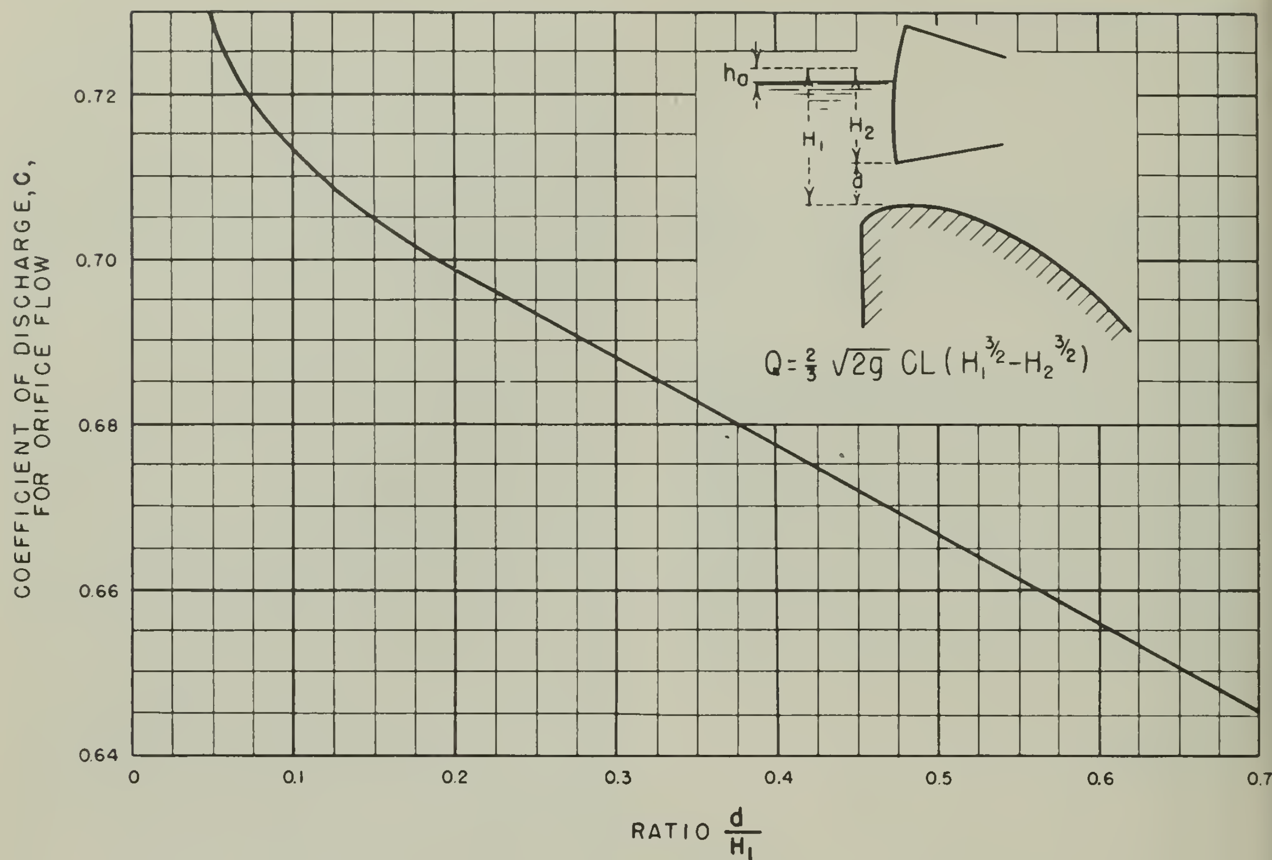


Figure 197. **Coefficient of discharge for flow under gates.**

tion the velocity and discharge will be $v+\Delta v$ and $Q+q(\Delta x)$, where q is the inflow per foot of length of weir crest. The momenta [3] at the two sections therefore will be:

$$\text{Upstream, } M_u=\frac{Qv}{g} \tag{8}$$

$$\text{Downstream, } M_d=\frac{[Q+q(\Delta x)]}{g}[v+\Delta v] \tag{9}$$

Subtracting equation (8) from equation (9):

$$\Delta M=\frac{Q(\Delta v)}{g}+\frac{q(\Delta x)}{g}[v+\Delta v] \tag{10}$$

Dividing by Δx:

$$\frac{\Delta M}{\Delta x}=\frac{Q(\Delta v)}{g(\Delta x)}+\frac{q}{g}[v+\Delta v] \tag{11}$$

[3] The weight of 1 cubic foot of water is taken as a unit of force to eliminate the necessity of multiplying all forces and momenta by 62.5 to convert them into pounds.

The rate of change of momentum with respect to time being v times the rate of change with respect to x, and considering the average velocity to be $\left[v+\frac{1}{2}(\Delta v)\right]$, equation (11) can be written:

$$\frac{\Delta M}{\Delta t}=\frac{Q(\Delta v)}{g(\Delta x)}\left[v+\frac{1}{2}(\Delta v)\right]+\frac{q}{g}[v+\Delta v]\left[v+\frac{1}{2}(\Delta v)\right] \tag{12}$$

As $\frac{\Delta M}{\Delta t}$ is the accelerating force, which is equal to the slope of the water surface $\frac{\Delta y}{\Delta x}$ times the average discharge, equation (12) becomes:

$$\frac{\Delta y}{\Delta x}\left[Q+\frac{1}{2}(\Delta Q)\right]=\frac{Q(\Delta v)}{g(\Delta x)}\left[v+\frac{1}{2}(\Delta v)\right]$$

$$+\frac{q}{g}[v+\Delta v]\left[v+\frac{1}{2}(\Delta v)\right] \tag{13}$$

from which the change in water surface elevation

$$\Delta y = \frac{Q}{g}\frac{\left[v+\frac{1}{2}(\Delta v)\right]}{\left[Q+\frac{1}{2}(\Delta Q)\right]}\left\{\Delta v+\frac{q(\Delta x)}{Q}\left[v+\Delta v\right]\right\} \quad (14)$$

If Q_1 and v_1 are values at the beginning of the reach and Q_2 and v_2 are the values at the end of the reach, the equation can be written:

$$\Delta y = \frac{Q_1}{g}\frac{(v_1+v_2)}{(Q_1+Q_2)}\left[(v_2-v_1)+\frac{v_2(Q_2-Q_1)}{Q_1}\right] \quad (15)$$

Similarly, the derivation can be developed so that:

$$\Delta y = \frac{Q_2}{g}\frac{(v_1+v_2)}{(Q_1+Q_2)}\left[(v_2-v_1)+\frac{v_1(Q_2-Q_1)}{Q_2}\right] \quad (16)$$

By use of equation (15) or (16), the water surface profile can be determined for any particular side channel by assuming successive short reaches of channel once a starting point is found. The solution of equation (15) or (16) is obtained by a trial and error procedure. For a reach of length Δx in a specific location, Q_1 and Q_2 will be known. If the depth at one end of the reach has been established, a trial depth at the other end of the reach can be found which will satisfy the indicated and computed values of Δy.

As in other water surface profile determinations, the depth of flow and the hydraulic characteristics of the flow will be affected by backwater influences from some control point, or by critical conditions along the reach of the channel under consideration. The selection of a control for starting the water surface profile computations is treated in the subsequent discussion.

When the bottom of the side channel trough is selected so that its depth below the hydraulic gradient is greater than the minimum specific energy depth, flow will be either at the subcritical or supercritical stage, depending on the relation of the bottom profile to critical slope or on the influences of a downstream control section. If the slope of the bottom is greater than critical and a control section is not established below the side channel trough, supercritical flow will prevail throughout the length of the channel. For this stage, velocities will be high and water depths will be shallow, resulting in a relatively high fall from the reservoir water level to the water surface in the trough. This flow condition is illustrated by profile B′ on figure 198. Conversely, if a control section is established downstream from the side channel trough to increase the upstream depths, the channel can be made to flow at the subcritical stage. Velocities at this stage will be less than critical and the greater depths will result in a smaller drop from the reservoir water surface to the side channel water surface profile. The condition of flow for subcritical depths is illustrated on figure 198 by water surface profile A′.

The effect of the fall distance from the reservoir to the channel water surface for each type of flow is depicted on figure 198(B). It can be seen that for the subcritical stage, the incoming flow will not develop high transverse velocities because of the low drop before it meets the channel flow, thus effecting a good diffusion with the water bulk in the trough. Since both the incoming velocities and the channel velocities will be relatively slow, a fairly complete intermingling of the flows will take place, thereby producing a comparatively smooth flow in the side channel. Where the channel flow is at the supercritical stage, the channel velocities will be high, and the intermixing of the high-energy transverse flow with the channel stream will be rough and turbulent. The transverse flows will tend to sweep the channel flow to the far side of the channel, producing violent wave action with attendant vibrations. It is thus evident that flows should be maintained at subcritical stage for good hydraulic performance. This can be achieved by establishing a control section downstream from the side channel trough.

The cross-sectional shape of the side channel trough will be influenced by the overflow crest on the one side and by the bank conditions on the opposite side. Because of turbulences and vibrations inherent in side channel flow, a side channel design is ordinarily not considered except where a competent foundation such as rock exists. The channel sides will, therefore, usually be a concrete lining placed on a slope and anchored directly to the rock. A trapezoidal cross section is the one most often employed for the side channel trough. The width of such a channel in relation to the depth should be considered. If the width to depth ratio is large, the depth of flow in the channel will be shallow, similar to that depicted by the cross section *abfg* on figure 199. It is evident that for this condition a poor diffusion

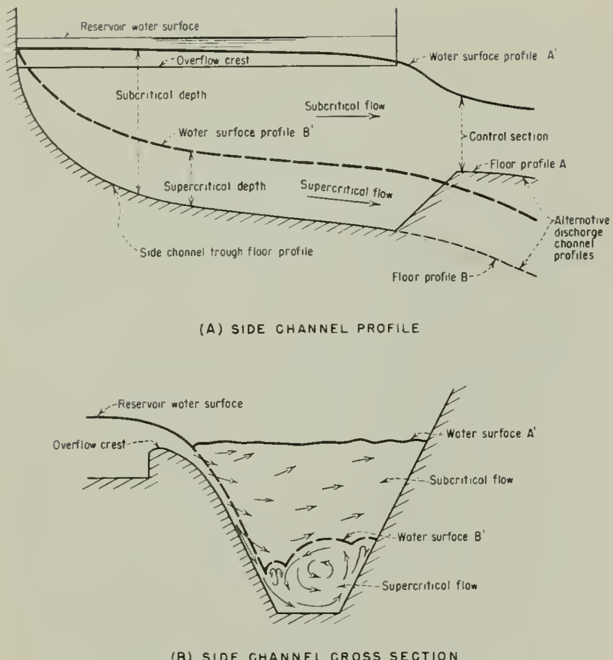


Figure 198. **Side channel flow characteristics.**

of the incoming flow with the channel flow will result. A cross section with a minimum width-depth ratio will provide the best hydraulic performance, indicating that a cross section approaching that depicted as *adj* on the figure would be the ideal choice both from the standpoint of hydraulics and economy. Minimum bottom widths are required, however, to avoid construction difficulties due to confined working space. Furthermore, the stability of the structure and the hillside which might be jeopardized by an extremely deep cut in the abutment must also be considered. Therefore, a minimum bottom width must be selected which is commensurate with both the practical and structural aspects of the problem.

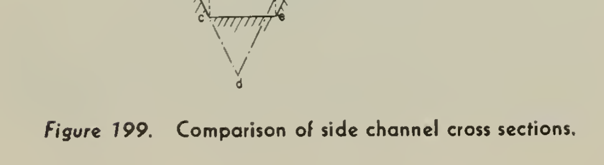


Figure 199. **Comparison of side channel cross sections.**

A control section downstream from the side channel trough is achieved by constricting the channel sides or elevating the channel bottom to produce a point of critical flow. Flows upstream from the control will be at the subcritical stage and will provide a maximum of depth in the side channel trough. The side channel bottom and control dimensions are then selected so that flow in the trough opposite the crest will be at the greatest depth possible without submerging the flow over the crest. Flow in the discharge channel downstream from the control will be the same as that in an ordinary channel or chute type spillway.

(b) *Design Example.*—A design example is provided to illustrate the procedures for determining the hydraulic design of a side channel spillway control structure. The problem is to design a side channel spillway 100 feet long (station 0+00 to station 1+00) to discharge a maximum of 2,000 second-feet. The spillway crest is at elevation 1000.0. The discharge per foot of length, q, is equal to $\frac{2,000}{100}=20$ second-feet. Assuming the crest coefficient, $C=3.6$, $H_o=\left(\frac{q}{C}\right)^{2/3}=3.1$ feet.

For the side channel trough, assume a trapezoidal section with ½:1 side slopes and a bottom width of 10 feet, whose rise in bottom profile is 1.0 foot in the 100 feet of channel length. (The slope of the channel profile is arbitrary; however, a relatively flat slope will provide greater depths and slower velocities and consequently will insure better intermingling of flows at the upstream end of the channel and avoid the possibility of accelerating or supercritical flows occurring in the channel for smaller discharges.) Also, assume that a control section is placed downstream from the side channel trough with its bottom at the same elevation as the bottom of the side channel floor at the downstream end, and that a transition is made from the ½:1 slopes of the trough section to a rectangular section at the control. Arbitrarily assume a datum for the control section bottom at elevation 100.0.

Then, the critical depth, d_c, for flow at the control is.

$$d_c=\sqrt[3]{\frac{q_1^2}{g}}$$

For this example:

$$q_1 = \frac{2,000}{10} = 200 \text{ second-feet per foot of width}$$

$$d_c = \sqrt[3]{\frac{200^2}{32.2}} = 10.75 \text{ feet}$$

$$v_c = \frac{q_1}{d_c} = \frac{200}{10.75} = 18.6 \text{ feet}$$

$$h_{v_c} = \frac{v_c^2}{2g} = \frac{18.6^2}{64.4} = 5.37 \text{ feet}$$

Assume a transition loss from the end of the side channel trough to the control section (to provide for losses due to contraction, to diffusion of the flows which were not effected in the side channel proper, and to friction losses) equal to 0.2 of the difference in velocity heads between the ends of the transition. The flow characteristics at the downstream end of the side channel are obtained from Bernoulli's theorem (appendix B). For figure 200, Bernoulli's theorem may be written as follows:

$$d_{(1+00)} + h_{v_{(1+00)}} = d_c + h_{v_c} + 0.2(h_{v_c} - h_{v_{(1+00)}})$$

This expression must be solved by trial and error. First, assume a value of $d_{(1+00)}$, and solve for $h_{v_{(1+00)}}$. If the use of these values does not result in a balanced equation, a new value must be assumed for $d_{(1+00)}$ and the process repeated. A value of 16.34 feet for $d_{(1+00)}$ was found to satisfy the equation as follows:

For $d_{(1+00)} = 16.34$, the area of flow at station 1+00 in the trapezoidal cross section with 10-foot bottom width and $\frac{1}{2}$:1 side slopes = 297 square feet.

$$v_{(1+00)} = \frac{2,000}{297} = 6.73 \text{ feet per second}$$

$$h_{v_{(1+00)}} = \frac{6.73^2}{64.4} = 0.70 \text{ foot}$$

$$0.2(h_{v_c} - h_{v_{(1+00)}}) = 0.2(5.37 - 0.70) = 0.93 \text{ foot.}$$

Substituting the values in the equation:

$$16.34 + 0.70 = 10.75 + 5.37 + 0.93$$

$$17.04 = 17.05 \text{ (A satisfactory check).}$$

With the hydraulic properties of the side channel at station 1+00 determined, the water surface profile along the side channel trough can be determined from equation (15). The trial-and-error computations are shown in table 20. The resulting water surface profile is shown on figure 200.

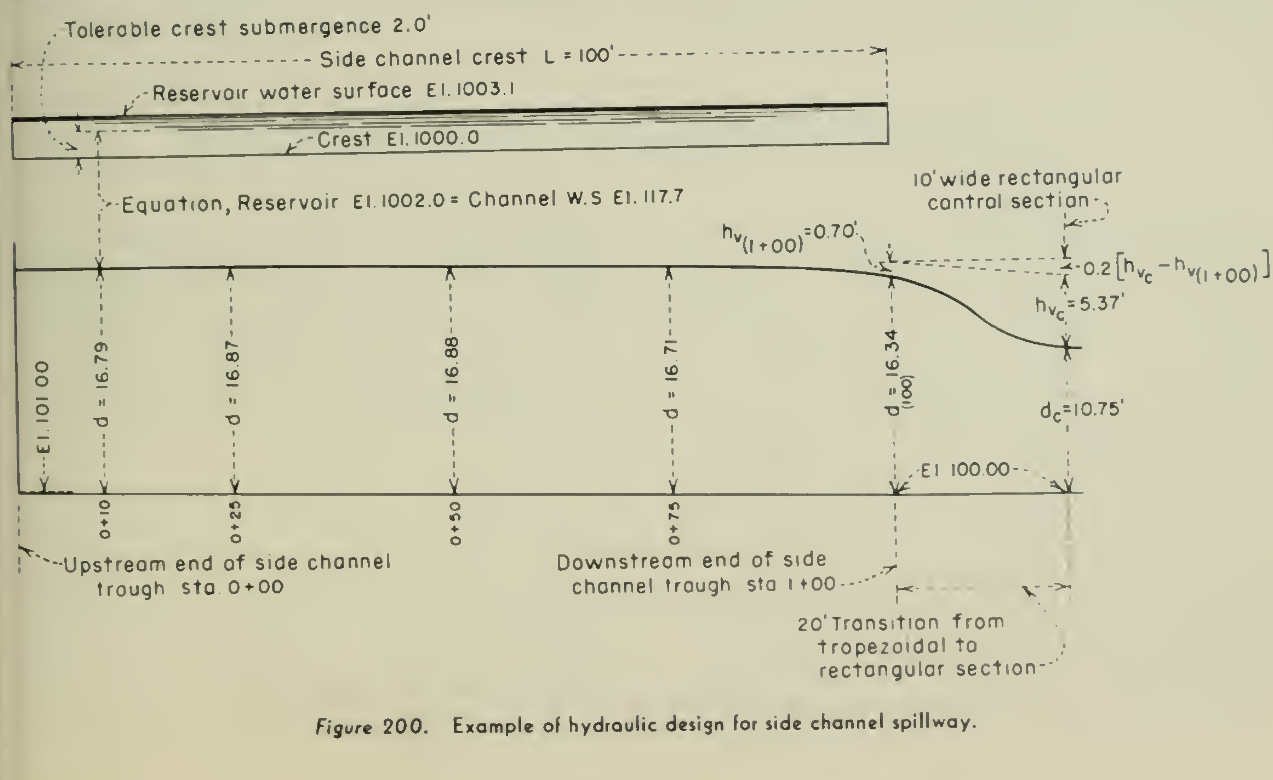


Figure 200. **Example of hydraulic design for side channel spillway.**

TABLE 20.—*Side channel spillway computations*

[Q=2,000 second-feet. Bottom width=10 feet. Side slopes=½:1. Bottom slope=1 foot in 100 feet]

Station	Δx	Elevation bottom	Trial Δy	Water surface elevation	d	A	Q	v	Q_1+Q_2	$\frac{Q_1}{g(Q_1+Q_2)}$	v_1+v_2	v_2-v_1	Q_2-Q_1	$\frac{Q_2-Q_1}{Q_1}$	$\frac{v_2(Q_2-Q_1)}{Q_1}$	(13)+(16)	Δy=(11) ×(12)×(17)	Remarks
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
1+00		100.0		116.34	16.34	297	2,000	6.73										
0+75	25	100.25	1.00	117.34	17.09	317	1,500	4.73	3,500	0.01332	11.46	2.00	500	0.333	2.24	4.24	0.64	Too low.
			.62	116.96	16.71	307		4.89			11.62	1.84				4.08	.63	OK.
0+50	25	100.50	.50	117.46	16.96	313	1,000	3.19	2,500	.01244	8.08	1.70	500	.50	2.44	4.14	.42	Too low.
			.42	117.38	16.88	311		3.22			8 11	1.67				4.11	.41	OK.
0+25	25	100.75	.30	117.68	16.93	313	500	1.60	1,500	.01036	4.82	1.62	500	1.00	3.22	4.84	.24	Too low.
			.24	117.62	16.87	311		1.61			4.83	1.61				4.83	.24	OK.
0+10	15	100.90	.10	117.72	16.82	310	200	.64	700	.00888	2.25	.97	300	1.50	2.41	3.38	.07	Too low.
			.07	117.69	16.79	309		.65			2.26	.96				3.37	.07	OK.

The channel profile is next fitted to the crest datum by relating the water surface profile to the reservoir water level. In order to obtain the assumed crest coefficient value of 3.6, excessive submergence of the overflow must be avoided. If it is assumed that a maximum of two-thirds submergence at the upstream end of the channel can be tolerated, the maximum water surface level in the channel will be ⅔ H_0 above the crest, or elevation 1002.0. Then at station 0+10, the channel datum water surface level elevation 117.7 will become elevation 1002.0, placing the channel floor level for station 0+00 at approximately elevation 985.3, and for station 1+00 at approximately elevation 984.3.

The design of the side channel control structure would be completed by designing the uncontrolled ogee crest by the methods shown in section 191, in order to obtain the crest coefficient value of 3.6 which was assumed.

Variations in the design can be made by assuming different bottom widths, different channel slopes, and varying control sections. A proper and economical design can usually be achieved after comparing several alternatives.

D. HYDRAULICS OF FREE-FLOW DISCHARGE CHANNELS

196. *General.*—Discharge generally passes through the critical stage in the spillway control structure and enters the discharge channel as supercritical or shooting flow. To avoid a hydraulic jump below the control, the flow must remain at the supercritical stage throughout the length of the channel. The flow in the channel may be uniform or it may be accelerated or decelerated, depending on the slopes and dimensions of the channel and on the total drop. Where it is desired to minimize the grade to reduce excavation at the upstream end of a channel, the flow might be uniform or decelerating, followed by accelerating flow in the steep drop leading to the downstream river level. Flow at any point along the channel will depend upon the specific energy, $(d+h_v)$, available at that point. This energy will equal the total drop from the reservoir water level to the floor of the channel at the point under consideration, less the head losses accumulated to that point. The velocities and depths of flow along the channel can be fixed by selecting the grade and the cross-sectional dimensions of the channel.

The velocities and depths of free surface flow in a channel, whether an open channel, a conduit, or a tunnel, conform to the principle of the conservation of energy as expressed by the Bernoulli's theorem, which states: "The absolute energy of flow at any cross section is equal to the absolute energy at a downstream section plus intervening losses of energy." As applied to

figure 201 this relationship can be expressed as follows:

$$\Delta Z + d_1 + h_{v_1} = d_2 + h_{v_2} + \Delta h_L \qquad (17)$$

When the channel grades are not too steep, for practical purposes the normal depth d_n can be considered equal to the vertical depth d. The term Δh_L includes all losses which occur in the reach of channel, such as friction, turbulence, impact, and transition losses. Since in most channels changes are made gradually, ordinarily all losses except those due to friction can be neglected. The friction loss can then be expressed as:

$$\Delta h_L = s.\Delta_L \qquad (18)$$

where s is the average friction slope expressed by either the Chezy or the Manning formula. For the reach ΔL, the head loss can be expressed as $\Delta h_L = \left(\frac{s_1+s_2}{2}\right)\Delta L$. From the Manning formula (equation (30), appendix B), $s = \left(\frac{vn}{1.486r^{2/3}}\right)^2$.

The coefficient of roughness, n, will depend on the nature of the channel surface. For conservative design the frictional loss should be maximized when evaluating depths of flow and minimized when evaluating the energy content of the flow. For determining depths of flow in a concrete-lined channel, a value of n of about 0.018 should be assumed in order to account for air swell, wave action, etc. For determining specific energies of flow needed for designing the dissipating device, a value of n of about 0.008 should be assumed.

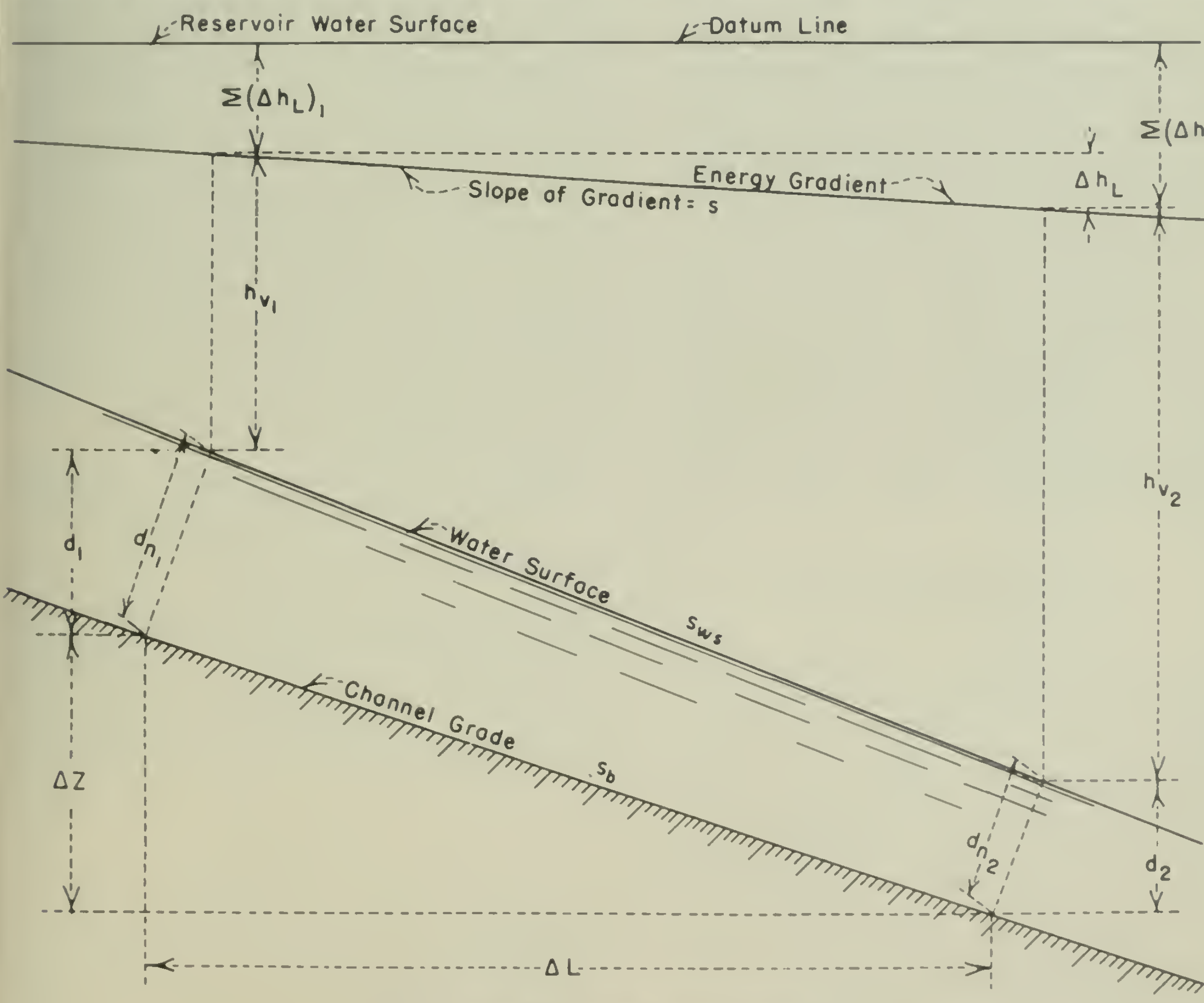


Figure 201. Flow in open channels.

Where only rough approximations of depths and velocities of flow in a discharge channel are desired, the total head loss $\Sigma(\Delta h_L)$ to any point along the channel might be expressed in terms of the velocity head. Thus, at any section the relationship can be stated: Reservoir water surface elevation minus floor grade elevation $=d+h_v+Kh_v$. For spillways with small drops, K can be assumed as approximately 0.2 for determining depths of flow and 0.1 or less for evaluating the energy of flow. Rough approximations of losses can also be obtained from figure B–5 (app. B).

197. Open Channels.—(a) *Profile.*—The profile of an open channel is usually selected to conform to topographic and geologic site conditions. It is generally defined as straight reaches joined by vertical curves. Sharp convex and concave vertical curves should be avoided to prevent unsatisfactory flows in the channel. Convex curves should be flat enough to maintain positive pressures and thus avoid the tendency for separation of the flow from the floor. Concave curves should have a sufficiently long radius of curvature to minimize the dynamic forces on the floor brought about by the centrifugal force which results from a change in the direction of flow.

To avoid the tendency for the water to spring away from the floor and thereby reduce the surface contact pressure, the floor shape for convex curvature should be made substantially flatter than the trajectory of a free-discharging jet issuing under a head equal to the specific energy of flow as it enters the curve. The curvature should approximate a shape defined by the equation:

$$-y=x\tan\theta+\frac{x^2}{K[4(d+h_v)\cos^2\theta]} \qquad (19)$$

where θ is the slope angle of the floor upstream from the curve. Except for the factor K, the equation is that of a free-discharging trajectory issuing from an inclined orifice. To assure positive pressure along the entire contact surface of the curve, K should be equal to or greater than 1.5.

For the concave curvature, the pressure exerted upon the floor surface by the centrifugal force of the flow will vary directly with the energy of the flow and inversely with the radius of curvature. An approximate relationship of these criteria can be expressed in the equations:

$$R=\frac{2qv}{p} \text{ or } R=\frac{2dv^2}{p} \qquad (20)$$

where:

R=the minimum radius of curvature measured in feet,

q=the discharge in second-feet per foot of width,

v=the velocity in feet per second,

d=the depth of flow in feet, and

p=the normal dynamic pressure exerted on the floor, in pounds per square foot.

An assumed value of $p=100$ will normally produce an acceptable radius; however, in no event should the radius be less than $10d$. For the reverse curve at the lower end of the ogee crest, radii of not less than $5d$ have been found acceptable.

(*b*) *Convergence and Divergence.*—The best hydraulic performance in a discharge channel is obtained when the confining sidewalls are parallel and the distribution of flow across the channel is maintained uniform. However, economy may dictate a channel section narrower or wider than either the crest or the terminal structure, thus requiring converging or diverging transitions to fit the various components together. Sidewall convergence must be made gradual to avoid cross waves, "ride ups" on the walls, and uneven distribution of flow across the channel. Similarly, the rate of divergence of the sidewalls must be limited or else the flow will not spread to occupy the entire width of the channel uniformly, which will result in undesirable flow conditions at the terminal structure.

The inertial and gravitational forces of streamlined kinetic flow in a channel can be expressed by the Froude number parameter, $\frac{v}{\sqrt{gd}}$. Variations from streamlined flow due to outside interferences which cause an expansion or a contraction of the flow also can be related to this parameter. Experiments have shown that an angular variation of the flow boundaries not exceeding that produced by the equation,

$$\tan\alpha=\frac{1}{3F} \qquad (21)$$

will provide an acceptable transition for either a contracting or an expanding channel. In this equation, $F=\frac{v}{\sqrt{gd}}$ and α is the angular variation

of the sidewall with respect to the channel centerline; v and d are the averages of the velocities and depths at the beginning and at the end of the transition. Figure 202 is a nomograph from which the tangent of the flare angle or the flare angle in degrees may be obtained for known values of depth and velocity of flow.

(e) *Channel Freeboard.*—In a channel conducting flow at supercritical stage, the surface roughness, wave action, air bulking and splash and spray are related to the velocity and energy content of the flow. The energy per foot of width, qh_v, expressed in terms of v and d is $\frac{v^3 d}{2g}$; therefore, the relationship of velocity and depth to the flow energy also can be expressed in terms of v and $\sqrt[3]{d}$. An empirical expression based on this relationship which gives a reasonable indication of desirable freeboard values is as follows:

$$\text{Freeboard (in feet)} = 2.0 + 0.025v\sqrt[3]{d} \qquad (22)$$

E. HYDRAULICS OF TERMINAL STRUCTURES

198. ***Deflector Buckets.***—Where the spillway discharge may be safely delivered directly to the river without providing a dissipating or stilling device, the jet is often projected beyond the structure by a deflector bucket or lip. Flow from these deflectors leaves the structure as a free-discharging upturned jet and falls into the stream channel some distance from the end of the spillway. The path the jet assumes depends on the energy of flow available at the lip and the angle at which the jet leaves the bucket.

With the origin of the coordinates taken at the end of the lip, the path of the trajectory is given by the equation:

$$y = x \tan\theta - \frac{x^2}{K[4(d+h_v)\cos^2\theta]} \qquad (23)$$

where:

θ = the angle of the edge of the lip with the horizontal, and

K = a factor, equal to 1 for the theoretical jet.

To compensate for loss of energy and velocity reduction due to the effect of air resistance, internal turbulences, and disintegration of the jet, a value for K of about 0.9 should be assumed.

The horizontal range of the jet at the level of the lip is obtained by making y in equation (23) equal to zero. Then:

$$x = 4K(d+h_v)\tan\theta\cos^2\theta$$
$$= 2K(d+h_v)\sin 2\theta$$

The maximum value of x will be equal to $2K(d+h_v)$ when θ is 45°. However, the angle of the lip is influenced by the bucket radius and the height of the lip above the bucket invert; ordinarily the exit angle is limited to not more than 30°.

The bucket radius should be made long enough to maintain concentric flow as the water moves around the curve. The rate of curvature must be limited similar to that of a vertical curve in a discharge channel (sec. 197), so that the floor pressures will not alter the streamline distribution

After C. Freeman

Figure 202. **Flare angle for divergent or convergent channels.**

of the flow. The minimum radius of curvature can be determined from equation (20), except that values of p not exceeding 500 pounds per square foot will produce values of the radius which have proved satisfactory in practice. However, the radius should not be less than five times the depth of water. Structurally, the cantilever bucket must be of sufficient strength to withstand this normal dynamic force in addition to the other applied forces.

199. *Hydraulic Jump Basins.*—(a) *General.*—Where the energy of flow in a spillway must be dissipated before the discharge is returned to the downstream river channel, the hydraulic jump basin is an effective device for reducing the exit velocity to a tranquil state. The jump which will occur in a stilling basin has distinctive characteristics and assumes a definite form, depending on the energy of flow which must be dissipated in relation to the depth of the flow. A comprehensive series of tests have been performed by the Bureau of Reclamation [9] for determining the properties of the hydraulic jump. The jump form and the flow characteristics can be related to the kinetic flow factor, $\frac{v^2}{gd}$, of the discharge entering the basin; to the critical depth of flow, d_c; or to the Froude number parameter, $\frac{v}{\sqrt{gd}}$. Forms of the hydraulic jump phenomena for various ranges of the Froude number are illustrated on figure 203.

When the Froude number of the incoming flow is equal to 1.0, the flow is at critical depth and a hydraulic jump cannot form. For Froude numbers from 1.0 up to about 1.7, the incoming flow is only slightly below critical depth, and the change from this low stage to the high stage flow is gradual and manifests itself only by a slightly ruffled water surface. As the Froude number approaches 1.7, a series of small rollers begin to develop on the surface, which become more intense with increasingly higher values of the number. Other than the surface roller phenomena, relatively smooth flows prevail throughout the Froude number range up to about 2.5. Stilling action for the range of Froude numbers from 1.7 to 2.5 is designated as form A on figure 203.

For Froude numbers between 2.5 and 4.5 an oscillating form of jump occurs, the entering jet intermittently flowing near the bottom and then along the surface of the downstream channel. This oscillating flow causes objectionable surface waves which carry considerably beyond the end of the basin. The action represented through this range of flows is designated as form B on figure 203.

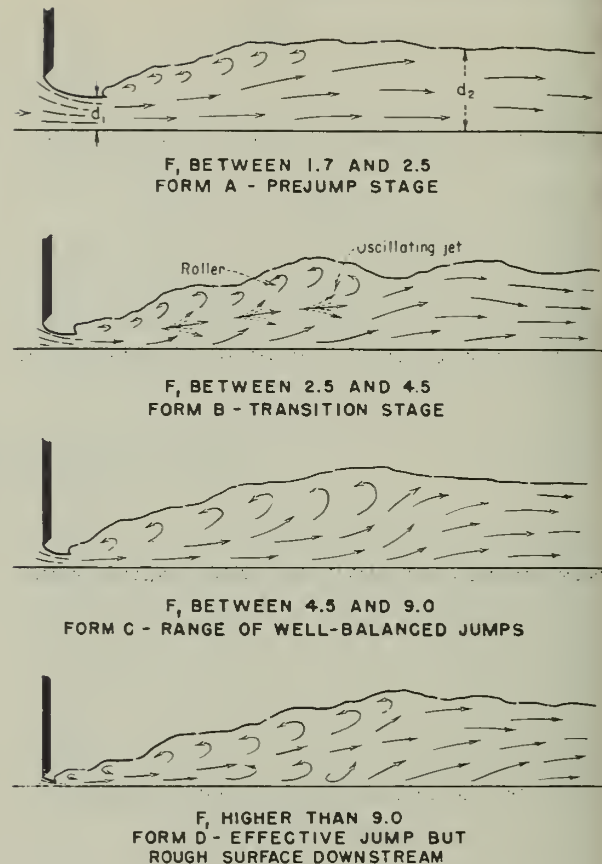


Figure 203. **Characteristic forms of hydraulic jump related to the Froude number.**

For the range of Froude numbers for the incoming flow between 4.5 and 9, a stable and well-balanced jump occurs. Turbulence is confined to the main body of the jump, and the water surface downstream is comparatively smooth. As the Froude number increases above 9, the turbulence within the jump and the surface roller becomes increasingly active, resulting in a rough water surface with strong surface waves downstream from the jump. Stilling action for the range of Froude numbers between 4.5 and 9 is designated as form C on figure 203, and that above 9 is designated as form D.

Figure 204 plots relationships of conjugate depths and velocities for the hydraulic jump in a rectangular channel. Also indicated on the figure

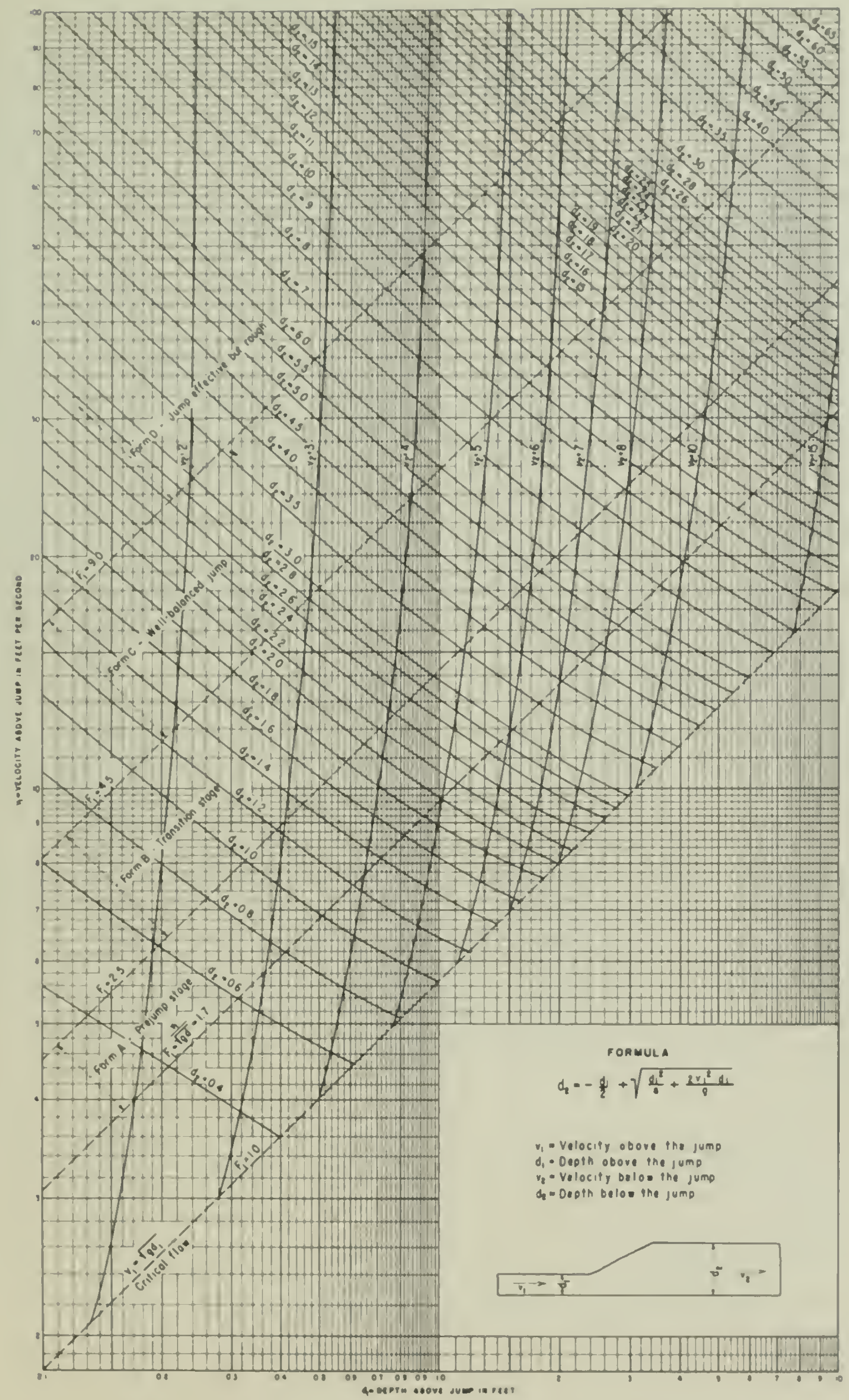


Figure 204. Relations between variables in hydraulic jump for rectangular channel.

are the ranges for the various forms of jump described above.

(b) *Basin Design in Relation to Froude Numbers.*—Stilling basin designs suitable to provide stilling action for the various forms of jump are described as follows:

(1) *Basins for Froude numbers less than 1.7.*—For a Froude number of 1.7 the conjugate depth d_2 is about twice the incoming depth, or about 40 percent greater than the critical depth. The exit velocity v_2 is about one-half the incoming velocity, or 30 percent less than the critical velocity. No special stilling basin is needed to still flows where the incoming flow Froude factor is less than 1.7, except that the channel lengths beyond the point where the depth starts to change should be not less than about $4d_2$. No baffles or other dissipating devices are needed.

(2) *Basins for Froude numbers between 1.7 and 2.5.*—Flow phenomena for basins where the incoming flow factors are in the Froude number range between 1.7 and 2.5 will be in the form designated as the prejump stage, as illustrated on figure 203. Since such flows are not attended by active turbulence, baffles or sills are not required. The basin should be sufficiently long to contain the flow prism while it is undergoing retardation. Conjugate depths and basin lengths shown in figure B–15 (app. B) will provide acceptable basins.

(3) *Basins for Froude numbers between 2.5 and 4.5.*—Jump phenomena where the incoming flow factors are in the Froude number range between 2.5 and 4.5 are designated as transition flow stage, since a true hydraulic jump does not fully develop. Stilling basins to accommodate these flows are the least effective in providing satisfactory dissipation, since the attendant wave action ordinarily cannot be controlled by the usual basin devices. Waves generated by the flow phenomena will persist beyond the end of the basin and must often be dampened by means apart from the basin.

Where a stilling device must be provided to dissipate flows for this range of Froude number, the basin shown on figure 205, which is designated as type I basin, has proved to be relatively effective for dissipating the bulk of the energy of flow. However, the wave action propagated by the oscillating flow cannot be entirely dampened. Auxiliary wave dampeners or wave suppressors must sometimes be employed to provide smooth surface flow downstream.

Because of the tendency of the jump to sweep out and as an aid in suppressing wave action, the water depths in the basin should be about 10 percent greater than the computed conjugate depth.

Often the need for utilizing this type of basin in design can be avoided by selecting stilling basin dimensions which will provide flow conditions which fall outside the range of transition flow. For example, with an 800-second-foot capacity spillway where the specific energy at the upstream end of the basin is about 15 feet and the velocity into the basin is about 30 feet per second, the Froude number will be 3.2 for a basin width of 10 feet. The Froude number can be raised to 4.6 by widening the basin to 20 feet. The selection of basin width then becomes a matter of economics as well as hydraulic performance.

(4) *Basins for Froude numbers higher than 4.5.*—For basins where the Froude number value of the incoming flow is higher than 4.5, a true hydraulic jump will form. The elements of the jump vary according to the Froude number as shown on figure B–15 (app. B). The installation of accessory devices such as blocks, baffles, and sills along the floor of the basin produce a stabilizing effect on the jump, which permits shortening the basin and provides a factor of safety against sweep-out due to inadequate tailwater depth.

The basin shown on figure 206, which is designated as a type II basin, can be adopted where incoming velocities do not exceed 50 feet per second. This basin utilizes chute blocks, impact baffle blocks, and an end sill to shorten the jump length and to dissipate the high-velocity flow within the shortened basin length. This basin relies on dissipation of energy by the impact blocks and also on the turbulence of the jump phenomena for its effectiveness. Because of the large impact forces to which the baffles are subjected by the impingement of high incoming velocities and because of the possibility of cavitation along the surfaces of the blocks and floor, the use of this basin must be limited to heads where the velocity does not exceed 50 feet per second.

Cognizance must be taken of the added loads placed upon the structure floor by the dynamic force brought against the upstream face of the baffle blocks. This dynamic force will approximate that of a jet impinging upon a plane normal

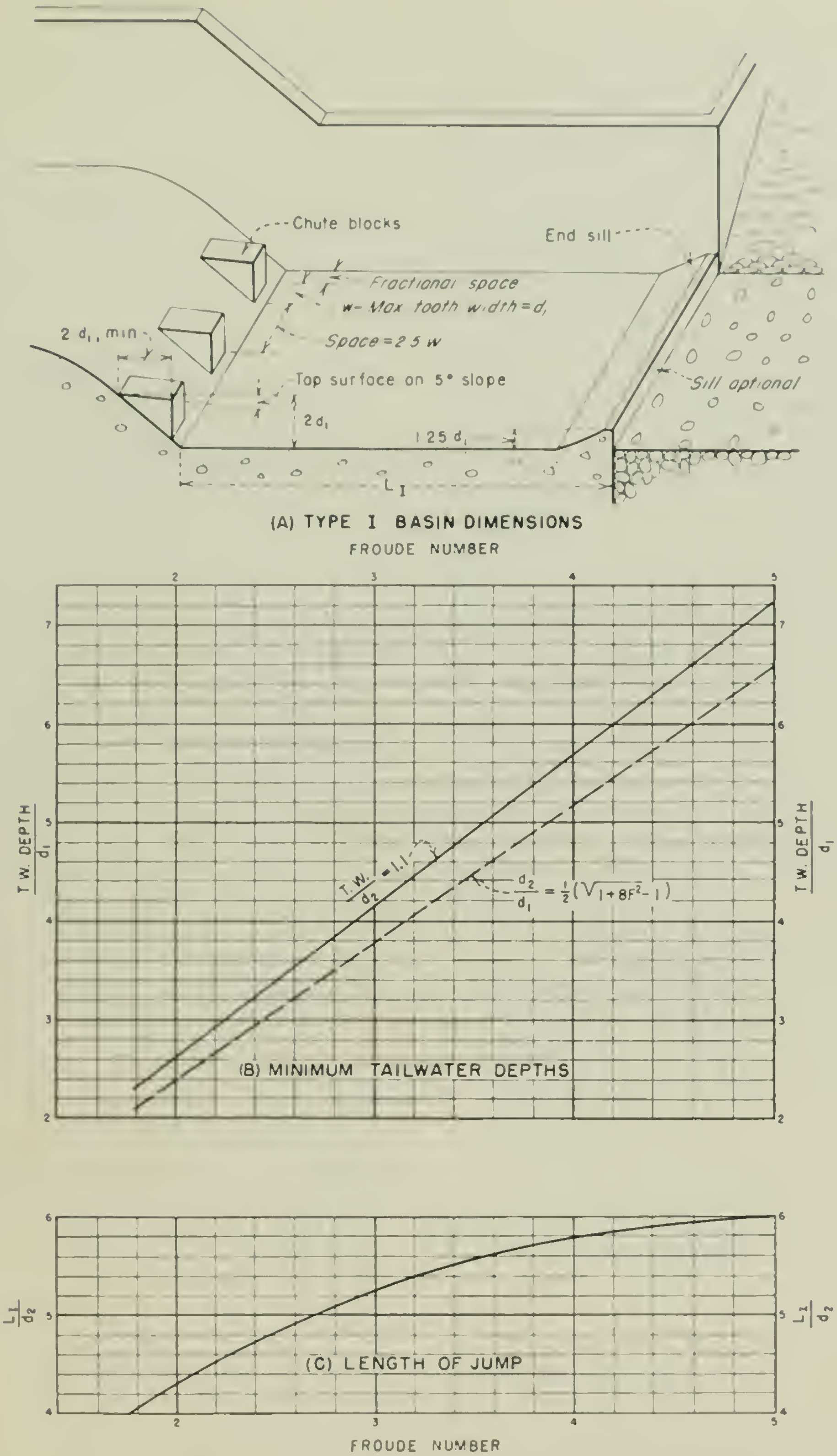


Figure 205. Stilling basin characteristics for Froude numbers between 2.5 and 4.5.

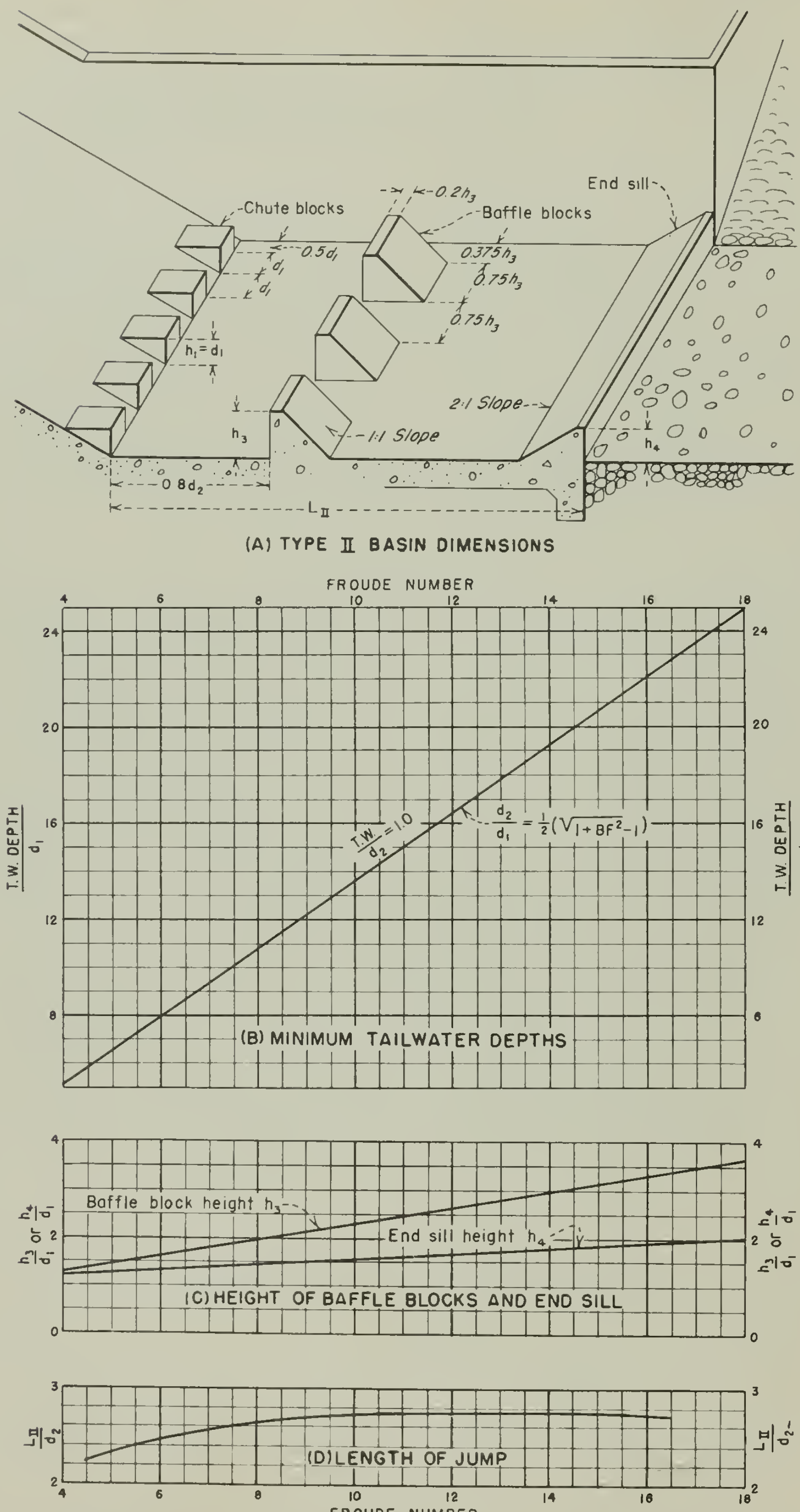


Figure 206. Stilling basin characteristics for Froude numbers above 4.5 where incoming velocity does not exceed 50 feet per second.

to the direction of flow. The force, in pounds per square foot, may be expressed by the formula:

$$\text{Force} = 2wA(d_1 + h_{v_1}) \qquad (24)$$

where:

w = the unit weight of water,

A = the area of the upstream face of the block, and

$(d_1 + h_{v_1})$ = the specific energy of the flow entering the basin.

Negative pressure on the back face of the blocks will further increase the total load. However, since the baffle blocks are placed a distance equal to $0.8d_2$ beyond the start of the jump, there will be some cushioning effect by the time the incoming jet reaches the blocks and the force will be less than that indicated by the above equation. If the full force computed by equation (24) is used, the negative pressure force may be neglected.

Where incoming velocities exceed 50 feet per second, or where impact baffle blocks are not employed, the basin designated as type III on figure 207 can be adopted. Because the dissipation is accomplished primarily by hydraulic jump action, the basin length will be greater than that indicated for the type II basin. However, the chute blocks and dentated end sill will still be effective in reducing the length from that which would be necessary if they were not used. Because of the reduced margin of safety against sweep-out, the water depth in the basin should be about 5 percent greater than the computed conjugate depth.

(c) *Rectangular Versus Trapezoidal Stilling Basin.*—The utilization of a trapezoidal stilling basin in lieu of a rectangular basin may often be proposed where economy favors sloped side lining over vertical wall construction. Model tests have shown, however, that the hydraulic jump action in a trapezoidal basin is much less complete and less stable than it is in the rectangular basin. In the trapezoidal basin the water in the triangular areas along the sides of the basin adjacent to the jump is not opposed by the incoming high-velocity jet. The jump, which tends to occur vertically, cannot spread sufficiently to occupy the side areas. Consequently, the jump will form only in the central portion of the basin, while areas along the outside will be occupied by upstream-moving flows which ravel off the jump or come from the lower end of the basin. The eddy or horizontal roller action resulting from this phenomenon tends to interfere and interrupt the jump action to the extent that there is incomplete dissipation of the energy and severe scouring can occur beyond the basin. For good hydraulic performance, the sidewalls of a stilling basin should be vertical or as near vertical as is practicable.

(d) *Basin Depths Versus Hydraulic Heads.*—The nomograph shown on figure 208 will aid in determining approximate basin depths for various basin widths and for various differences between reservoir and tailwater levels. Plottings are shown for the condition of no loss of head to the upstream end of the stilling basin, and for 10, 20, and 30 percent loss. (These plottings are shown on the nomographs as scales A, B, C, and D, respectively.) The required conjugate depths, d_2, will depend on the specific energy available at the entrance of the basin, as determined by the procedure discussed in section 196. Where the specific energy is known, the head loss in the channel upstream can be related to the velocity head, the percentage loss can be determined, and the approximate conjugate depth can be read from the nomograph. Where head losses have not been computed, a quick approximation of the head losses can be obtained from figure B–5 (app. B). Where only a rough determination of basin depths is needed, the choice of the loss ot be applied for various spillway designs may be generalized as follows:

(1) For a design of an overflow spillway where the basin is directly downstream from the crest, or where the chute is not longer than the hydraulic head, consider no loss of head.

(2) For a design of a channel spillway where the channel length is between one and five times the hydraulic head, consider 10 percent loss of head.

(3) For a design of a spillway where the channel length exceeds five times the hydraulic head, consider 20 percent loss of head.

The nomograph on figure 208 gives values of the conjugate depth of the hydraulic jump. Tailwater depths for the various types of basin described in section 199 should be increased as noted in that section.

(e) *Tailwater Considerations.*—Determination of the tailwater rating curve, which gives the stage-discharge relationship of the natural stream below the dam, is discussed in appendix B, part B. Tailwater rating curves for the regime of river

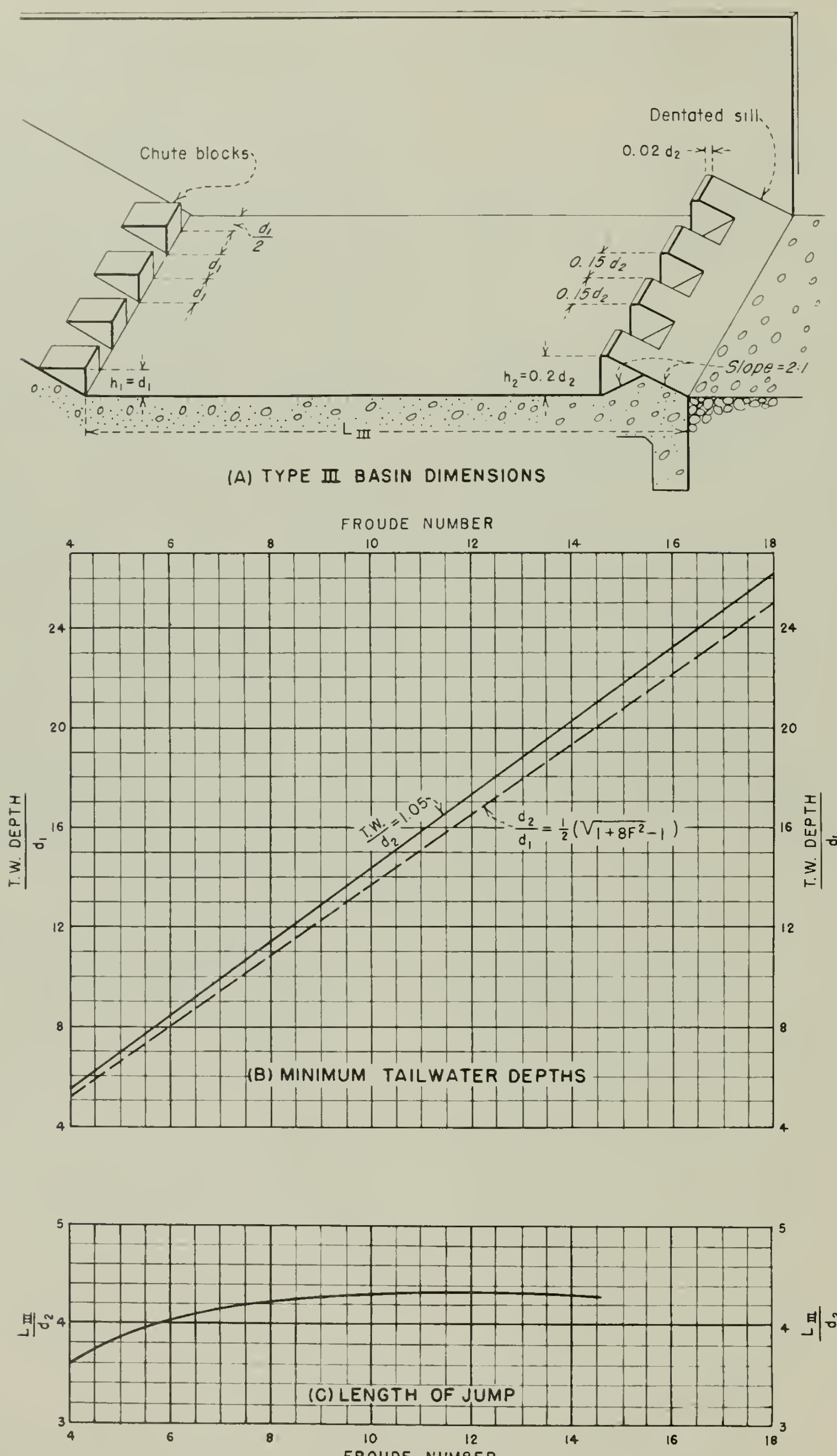


Figure 207. **Stilling basin characteristics for Froude numbers above 4.5.**

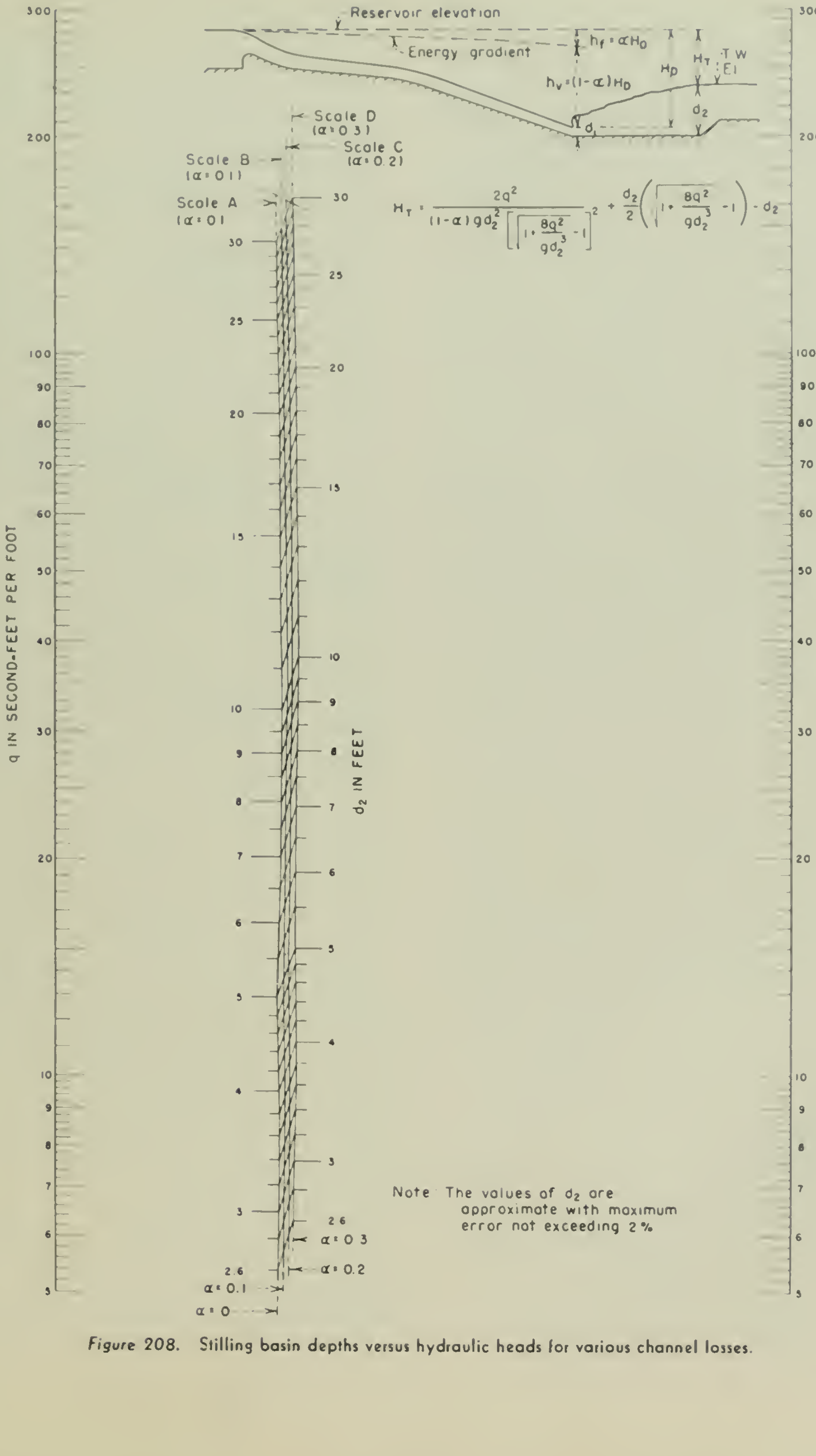


Figure 208. Stilling basin depths versus hydraulic heads for various channel losses.

below a dam are fixed by the natural conditions along the stream and ordinarily cannot be altered by the spillway design or by the release characteristics. As discussed in section 185(d), retrogression or aggradation of the river below the dam, which will affect the ultimate stage-discharge conditions, must be recognized in selecting the tailwater rating curve to be used for stilling basin design. Usually river flows which approach the maximum design discharges have never occurred, and an estimate of the tailwater rating curve must either be extrapolated from known conditions or computed on the basis of assumed or empirical criteria. Thus, the tailwater rating curve at best is only approximate, and factors of safety to compensate for variations in tailwater must be included in the design.

For a jump-type stilling basin, downstream water levels for various discharges must conform to the tailwater rating curve, and the basin floor level must therefore be selected to provide jump depths which most nearly agree with the tailwater depths. For a given basin design, the tailwater depth for each discharge seldom corresponds to the conjugate depth needed to form a perfect jump. Thus, the relative shapes and relationships of the tailwater curve to the depth curve will determine the required minimum depth to the basin floor. This is illustrated on figure 209. The tailwater rating curve is shown in (A) as curve 1, and a conjugate depth versus discharge curve for a basin of certain width is represented by curve 3. Since the basin must be made deep enough to provide for full conjugate depth (or some greater depth to include a factor of safety) at the maximum spillway design discharge, the curves will intersect at point D. For lesser discharges the tailwater depth will be greater than the required conjugate depth, thus providing an excess of tailwater which is conducive to the formation of a so-called drowned jump. (With the drowned jump condition, instead of achieving good jump-type dissipation by the intermingling of the upstream and downstream flows, the incoming jet plunges to the bottom and carries along the entire length of the basin floor at high velocity.) If the basin floor is made higher than indicated by the position of curve 3 on the figure, the depth curve and tailwater rating curve will intersect to the left of point D, thus indicating an excess of tailwater for smaller discharges and a deficiency of tailwater for higher discharges.

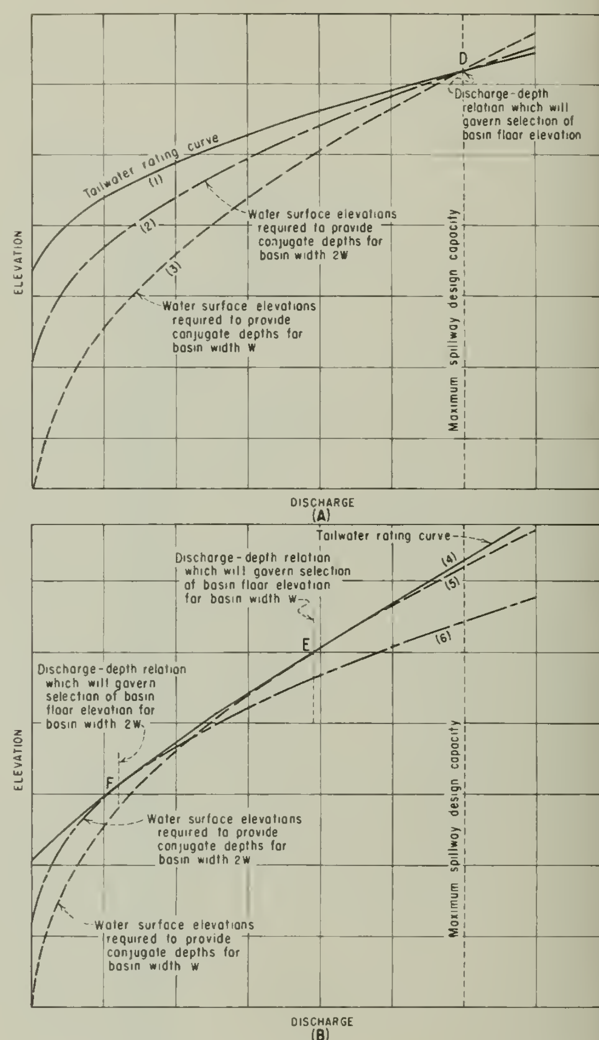


Figure 209. **Relationships of conjugate depth curves to tailwater rating curves.**

As an alternative to the selected basin which is represented by curve 3, a wider basin might be considered for which the conjugate depth curve 2 will apply. This design will provide a shallower basin, in which the ideal jump depths will more nearly match the tailwater depths for all discharges. The choice of basin widths, of course, involves consideration of economics, as well as hydraulic performance.

Where a tailwater rating curve shaped similar to that represented by curve 4 on figure 209(B) is encountered, the level of the stilling basin floor must be determined for some discharge other than the maximum design capacity. If the tailwater curve were made to intersect the required water

surface elevation at the maximum design capacity, as in figure 209(A), there would be insufficient tailwater depth for most smaller discharges. In this case the basin floor elevation is selected so that there will be sufficient tailwater depth for all discharges. For the basin of width W, whose required tailwater depth is represented by curve 5, the position of the floor would be selected so that the two curves would coincide at the discharge represented by point E on the figure. For all other discharges the tailwater depth will be in excess of that needed for forming a satisfactory jump. Similarly, if a basin width of $2W$ were considered, the basin floor level would be selected so that curve 6 would intersect the tailwater curve at point F. Here also, the selection of basin widths should be based on economic aspects as well as hydraulic performance.

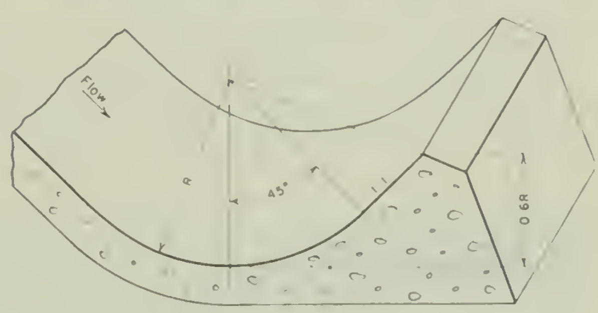
Where exact conjugate depth conditions for forming the jump cannot be attained, the question of the relative desirability of having insufficient tailwater as compared to having excessive tailwater should be considered. With insufficient tailwater the back pressure will be deficient and sweep-out of the basin will occur. With an excess of tailwater the jump will be formed and energy dissipation within the basin will be quite complete until the drowned jump phenomenon becomes critical. Chute blocks, baffles, and end sills will further assist in energy dissipation, even with a drowned jump.

(f) *Stilling Basin Freeboard.*—Freeboard is ordinarily provided so that the stilling basin walls will not be overtopped by surges, splash and spray, and wave action set up by the turbulence of the jump. The surface roughness of the flow is related to the energy dissipated in the jump and to the depth of flow in the basin. The following empirical expression provides values which have proved to be satisfactory for most basins:

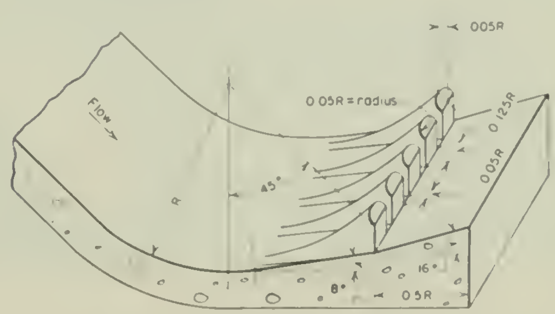
$$\text{Freeboard in feet} = 0.1(v_1 + d_2) \qquad (25)$$

200. *Submerged Bucket Dissipators.*—When the tailwater depth is too great for the formation of a hydraulic jump, dissipation of the high energy of flow can be effected by the use of a submerged bucket deflector. The hydraulic behavior in this type of dissipator is manifested primarily by the formation of two rollers; one is on the surface moving counterclockwise and is contained within the region above the curved bucket, and the other is a ground roller moving in a clockwise direction and is situated downstream from the bucket. The movements of the rollers, along with the intermingling of the incoming flows, effectively dissipate the high energy of the water and prevent excessive scouring downstream from the bucket.

Two types of roller bucket have been developed and model tested [10, 11]. Their shape and dimensional arrangements are shown on figure 210. The general nature of the dissipating action for each type is represented on figure 211. Hydraulic action of the two buckets has the same characteristics, but distinctive features of the flow differ to the extent that each has certain limitations. The high-velocity flow leaving the deflector lip of the solid bucket is directed upward. This creates a high boil on the water surface and a violent ground roller moving clockwise downstream from the bucket. This ground roller continuously pulls loose material back towards the lip of the bucket and keeps some of the intermingling material in a constant state of agitation. A typical scour pattern which results from this action is shown on figure 212. In the slotted bucket the high-velocity jet leaves the lip at a flatter angle, and only a part of the high-velocity flow finds its way to the surface. Thus, a less violent surface boil occurs and there is a better dispersion of flow in the



(A) SOLID BUCKET



(B) SLOTTED BUCKET

Figure 210. Submerged buckets.

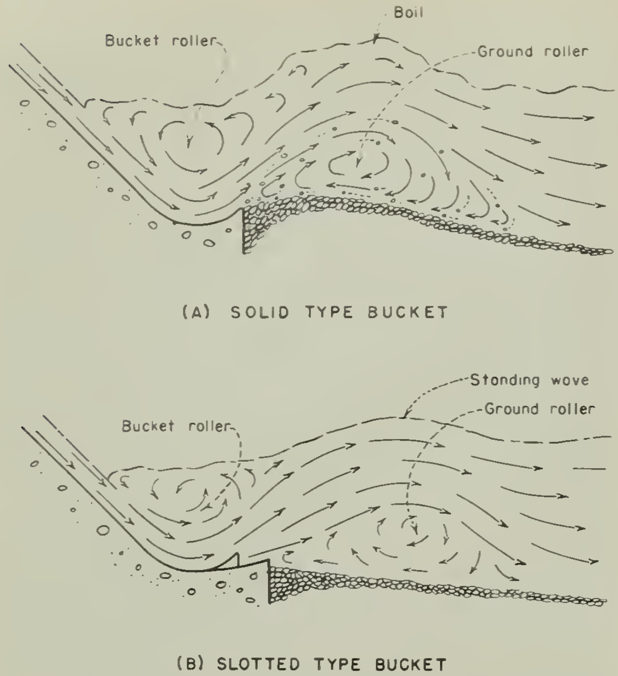


Figure 211. **Hydraulic action in solid and slotted buckets.**

region above the ground roller which results in less concentration of high-energy flow throughout the bucket and a smoother downstream flow.

Use of a solid bucket dissipator may be objectionable because of the abrasion on the concrete surfaces caused by material which is swept back along the lip of the deflector by the ground roller. In addition, the more turbulent surface roughness induced by the severe surface boil carries farther down the river, causing objectionable eddy currents which contribute to riverbank sloughing. Although the slotted bucket provides better energy dissipation with less severe surface and streambed disturbances, it is more sensitive to sweep-out at lower tailwaters and is conducive to a diving and scouring action at excessive tailwaters. This is not the case with the solid bucket. Thus, the tailwater range which will provide good performance with the slotted bucket is much narrower than that of the solid bucket. A solid bucket dissipator should not be used wherever the tailwater limitations of the slotted bucket can be met. Therefore, only the design of the slotted bucket will be discussed.

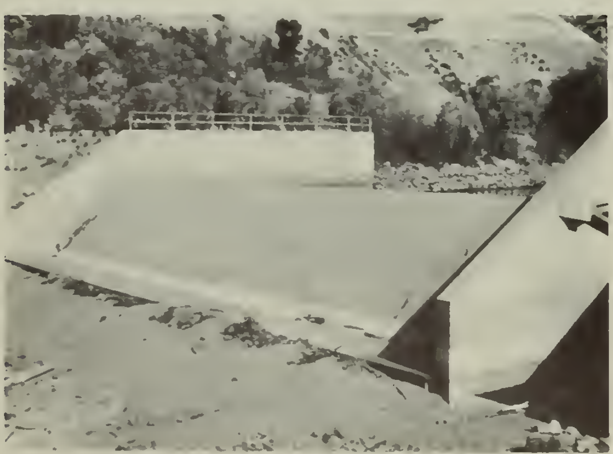


Figure 212. **Scour pattern downstream from a solid bucket dissipator for an ogee overflow crest—Murdock Diversion Dam in Utah.**

Flow characteristics of the slotted bucket are illustrated on figure 213. For deficient tailwater depths the incoming jet will sweep the surface roller out of the bucket and will produce a high-velocity flow downstream, both along the water surface and along the riverbed. This action is depicted as stage (A) on figure 213. As the tailwater depth is increased, there will be a depth at which instability of flow will occur, where sweep-out and submergence will alternately prevail. To obtain continuous operation at the submerged stage, the minimum tailwater depth must be above this instable state. Flow action within the acceptable operating stage is depicted as stage (B) on figure 213.

When the tailwater becomes excessively deep, the phenomenon designated as diving flow will occur. At this stage the jet issuing from the lip of the bucket will no longer rise and continue along the surface but intermittently will become depressed and dive to the riverbed. The position of the downstream roller will change with the change in position of the jet. It will occur at the surface when the jet dives and will form along the river bottom as a ground roller when the jet rides the surface. Scour will occur in the streambed at the point of impingement when the jet dives but will be filled in by the ground roller when the jet rides. The characteristic flow pattern for the diving stage is depicted in (C) and (D) of figure 213. Maximum tailwater depths must be limited to forestall the diving flow phenomenon.

The design of the slotted bucket involves determination of the radius of curvature of the bucket and the allowable range of tailwater depths. These criteria, as determined from experimental results, are plotted on figure 214 in relation to the

Froude number parameter. The Froude number values are for flows at the point where the incoming jet enters the bucket. Symbols and criteria are defined on figure 215.

201. *Examples of Designs of a Stilling Basin and an Alternate Submerged Bucket Dissipator.*—The designs of a stilling basin and of a submerged bucket dissipator are best explained by means of examples. Consider that it is required to make comparative designs of a stilling basin and of a submerged bucket dissipator for an overflow dam whose maximum discharge is 2,000 second-feet and whose controlling dimensions and tailwater conditions are as shown on figure 216.

For a first trial design, assume a crest length of 20 feet. The criteria for different discharges are then as follows:

Total discharge, Q, second-feet	2,000	1,000	500
Discharge per foot, q, second-feet	100	50	25
Assumed coefficient of discharge, C	3.9	3.7	3.5
Head in feet on crest, $H_e=\left(\frac{q}{C}\right)^{2/3}$	8.7	5.7	3.7
Reservoir water level, elevation	1008.7	1005.7	1003.7
Tailwater level, elevation	985.0	981.0	978.0
Reservoir water level minus tailwater level, feet	23.7	24.7	25.7
Velocity head at tailwater level, h_{v_t}, feet (assuming no loss of specific energy)	23.7	24.7	25.7
Velocity of flow in feet per second at tailwater level, $v_t=\sqrt{2gh_{v_t}}$	39.1	39.9	40.7
Depth of flow in feet at tailwater level, $d_t=\frac{q}{v_t}$	2.56	1.25	.61
Froude number at tailwater level, $F_t=\frac{v_t}{\sqrt{gd_t}}$	4.3	6.3	9.2
Specific energy at tailwater level, $d_t+h_{v_t}$	26.3	25.9	26.3

Table 21 shows the computations for a hydraulic jump basin design. Conjugate depths and the required apron elevation for the various discharges are calculated to determine the critical condition. The lowest apron elevation is for the 2,000-second-foot discharge. The Froude number of 6.2 and the incoming velocity not exceeding 50 feet per second, determine that the type II stilling basin shown on figure 206 should be used for this design. The basin length will be 42 feet and the apron elevation will be 968.3.

For the submerged slotted bucket design, the minimum bucket radius for the maximum discharge is determined by use of figure 214. For a Froude number at tailwater level of 4.3, the minimum radius is $0.42(d_t+h_{v_t})=0.42\times 26.3=11.0$ feet. In this instance the riverbed slopes up, and the use of figure 214 results in the following

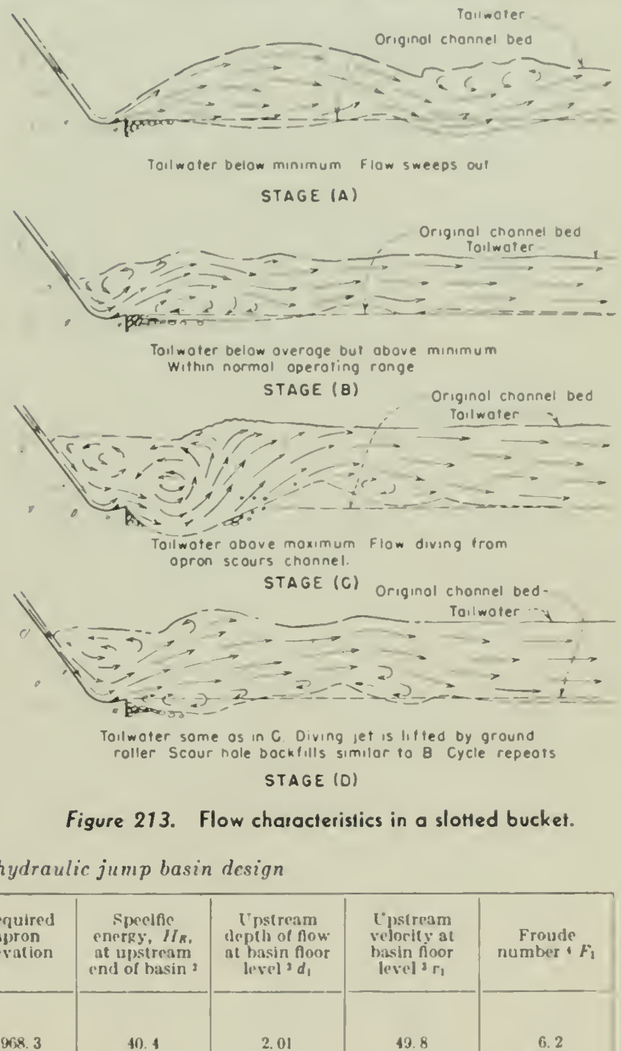


Figure 213. **Flow characteristics in a slotted bucket.**

TABLE 21.—*Computations for hydraulic jump basin design*

Discharge, Q, second-feet	Discharge per foot, q, second-feet	Reservoir level minus tailwater, feet	Conjugate depth, d_2 feet [1]	Tailwater elevation	Required apron elevation	Specific energy, H_E, at upstream end of basin [2]	Upstream depth of flow at basin floor level [3] d_1	Upstream velocity at basin floor level [3] v_1	Froude number [4] F_1
2,000	100	23.7	16.7	985.0	968.3	40.4	2.01	49.8	6.2
1,000	50	24.7	11.8	981.0	969.2	36.5	1.05	47.6	8.1
500	25	25.7	8.6	978.0	969.4	34.3	0.54	46.3	11.1

[1] From figure 208, assuming no loss in specific energy.

[2] H_E = Reservoir water surface minus apron elevation, assuming no loss in specific energy.

[3] $H_E=d_1+\frac{v_1^2}{2g}$.

[4] $F_1=\frac{v_1}{\sqrt{gd_1}}$.

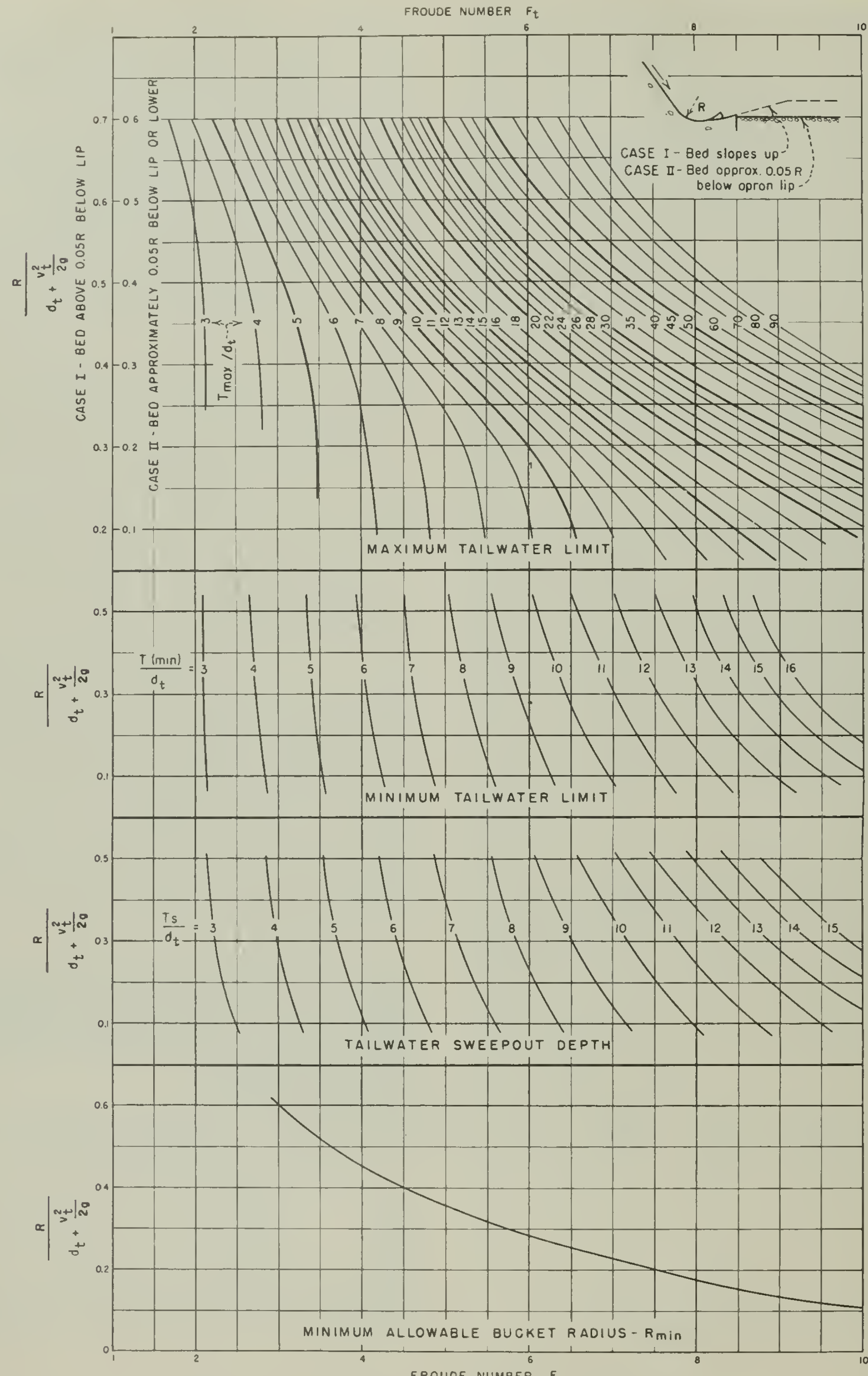


Figure 214. **Limiting criteria for slotted bucket design.**

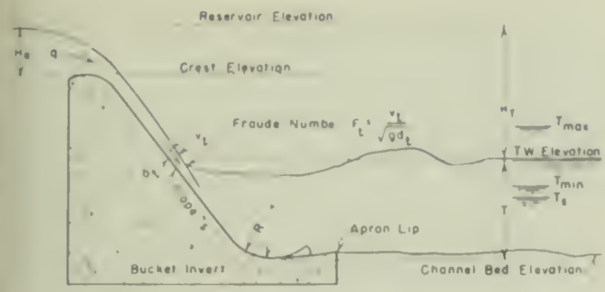


Figure 215. **Definition of symbols—submerged buckets.**

values for the maximum and minimum tailwater for a Froude number of 4.3 and a $\frac{R}{d_t+h_{v_t}}$ value of 0.42:

$$T_{max}=7.5\ d_t=7.5\times 2.56=19.2 \text{ feet}$$

$$T_{min}=6.5\ d_t=6.5\times 2.56=16.6 \text{ feet}$$

An average tailwater depth of 18 feet will place the bucket invert at elevation 985.0−18.0=967.0. It is now necessary to check the radius and tailwater conditions for less than maximum flows to determine if the design is satisfactory throughout the range of discharge.

For a unit discharge of 50 second-feet, the minimum radius for F_t of 6.3 is $0.26(d_t+h_{v_t})=0.26\times 25.9=6.8$ feet. Therefore, the minimum radius of 11.0 feet determined for the maximum discharge will govern. The maximum and minimum tailwater values for a Froude number of 6.3 and an $\frac{R}{d_t+h_{v_t}}$ value of $\frac{11}{25.9}$ or 0.42 are:

$$T_{max}=20.0\ d_t=20.0\times 1.25=25.0 \text{ feet}$$
$$T_{min}=10.1\ d_t=10.1\times 1.25=12.6 \text{ feet}$$

The bucket invert level at elevation 967 as determined for the maximum discharge will provide a tailwater depth of 981.0−967.0=14 feet, which is within the safe limit for producing satisfactory roller action.

The same procedure should be followed to verify that satisfactory roller action will result for a unit discharge of 25 second-feet. In this case the minimum radius of 11.0 feet determined for the maximum discharge was found to govern. The T_{max} and T_{min} were found to be 50 feet and 10.4 feet, respectively, compared to the 11 feet of tailwater depth provided by the invert elevation placed at 967.0 feet. It may now be considered that the design based on maximum discharges will be satisfactory for all lower discharges.

If a wider range of safe tailwater depths is desired, the radius of curvature of the bucket can be increased. Thus, for a bucket radius of 12 feet, for the maximum discharge, $T_{min}=6.5d_t=16.6$, and $T_{max}=8.5d_t=22.5$ feet. An average tailwater depth of 20 feet, placing the bucket invert at elevation 965, will provide more leeway for tailwater variations.

202. *Impact Type Stilling Basins.* An impact type of energy dissipator has been developed [12] which is an effective stilling device even with deficient tailwater where the discharge is relatively small and the incoming velocity into a basin does exceed 30 feet per second. This basin can be used with either an open chute or a closed conduit structure. The design shown on figure 217 has been proved for discharges up to about 400 second-feet; for larger discharges multiple basins could be placed side by side.

Dissipation is accomplished by the impact of the incoming jet on the vertical hanging baffle, and by eddies which are formed from the changed direction of the jet after it strikes the baffle. Best hydraulic action is obtained when the tailwater height approaches but does not exceed a level halfway up the height of the baffle. For proper performance, the bottom of the baffle should be placed at the same level as the invert of the upstream channel or pipe.

The general arrangement of the basin and the dimensional requirements for various discharges are shown on figure 217. Figure 218 shows an impact type stilling basin operating at about 80 percent of its designed capacity. This type of basin is subjected to large dynamic forces and turbulences which must be considered in the

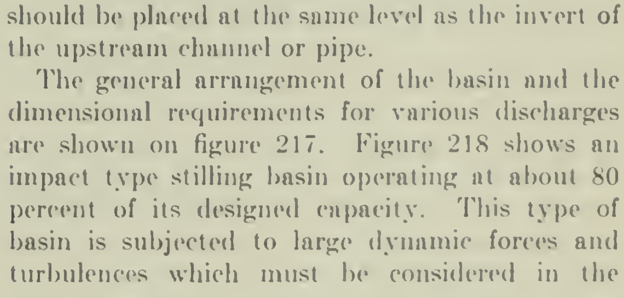
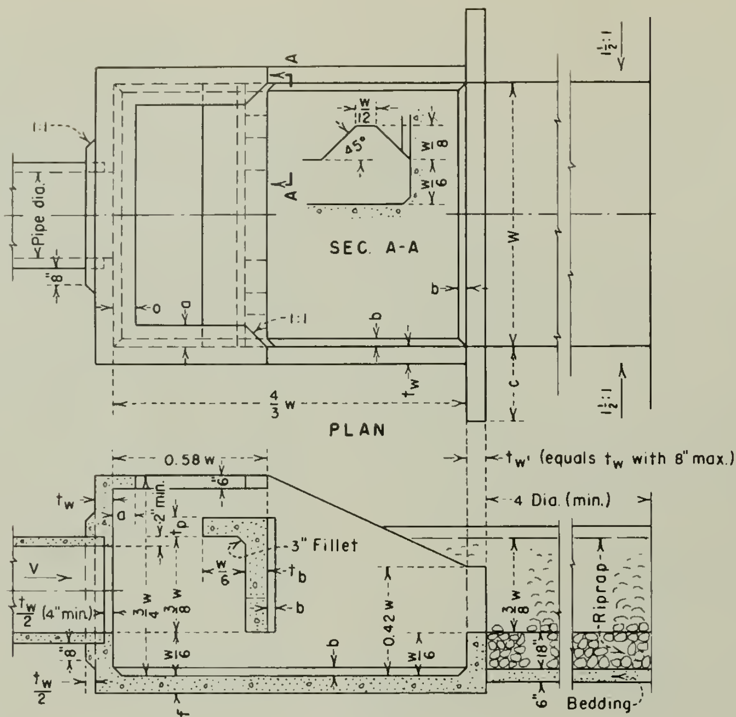
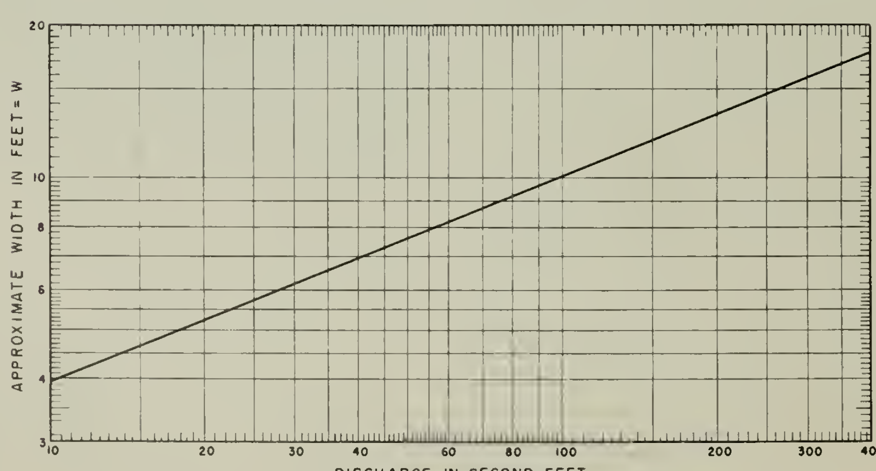


Figure 216. **Example of design of stilling device for overflow spillway.**



Q	a	b	c	t_w	t_f	t_b	t_p
100	9"	3"	3'-0"	8"	8"	9"	8"
200	12"	4"	3'-0"	10"	11"	10"	8"
300	14"	6"	3'-0"	12"	12"	12"	8"
400	16"	6"	3'-0"	12"	13"	12"	8"
Suggested minimum thickness of concrete is 6"							

SUGGESTED CONCRETE DIMENSIONS



HYDRAULIC DIMENSIONS

Figure 217. **Dimensional criteria for impact type stilling basin.**

structural design. The structure must be made sufficiently stable to resist sliding against the impact load on the baffle wall. The entire structure must resist the severe vibrations inherent in this type of device, and the individual structural members must be sufficiently strong to withstand the large dynamic loads.

Riprapping should be provided along the bottom and sides adjacent to the structure to avoid the tendency for scour of the outlet channel downstream from the end sill when a shallow tailwater exists. Downstream wingwalls placed at 45° may also be effective in reducing scouring tendencies and flow concentrations downstream.

Figure 218. An impact type stilling basin in operation.

203. *Plunge Pools.*—When a free-falling overflow nappe drops vertically into a pool in a riverbed, a plunge pool will be scoured to a depth which is related to the height of the fall, the depth of tailwater, and the concentration of the flow [13]. Depths of scour are influenced initially by the erodibility of the stream material or the bedrock and by the size or the gradation of sizes of any armoring material in the pool. However, the armoring or protective surfaces of the pool will be progressively reduced by the abrading action of the churning material to a size which will be scoured out and the ultimate scour depth will, for all practical considerations, stabilize at a limiting depth irrespective of the material size. An empirical approximation based on experimental data has been developed by Veronese [14] for limiting scour depths, as follows:

$$d_s = 1.32\, H_T^{0.225}\, q^{0.54} \qquad (26)$$

where,

d_s= the maximum depth of scour below tailwater level in feet,

H_T= the head from the reservoir to tailwater levels in feet, and

q= the discharge in second-feet per foot of width.

F. HYDRAULICS OF SPILLWAYS

204. *Free Overfall (Straight Drop) Spillways.*—(a) *General.*—The hydraulic problems of the free overfall spillway are concerned with the characteristics of the control and with the dissipation of flow in the downstream basin. Flow over the control ordinarily is free discharging; air is admitted to the underside of the nappe to avoid the jet being depressed by reduced underneath pressure. Dissipation of the flow in the downstream basin may be obtained by the hydraulic jump, by impact and turbulence induced in a basin with impact blocks, or by a slotted grating dissipator installed immediately downstream from the control.

The control may be either sharp crested to provide a fully contracted vertical jet, broad crested to effect a fully suppressed jet, or shaped to increase the crest efficiency. Coefficients of discharge will approximate those indicated in section 190. The sides of the control usually are arranged to allow for full side contraction, in order to provide side space for the access of air to the underside of the nappe. This contraction is effected by providing square abutment headwalls or by installing square-cornered vertical offsets along the piers or walls opposite the crest. The effective length of the crest is then determined according to

equation (4) where K_p and K_a will approximate 0.20.

The dimensions of the stilling basin for the free overfall spillway can be related to two independent variables; namely, the drop distance Y and the unit discharge q. These variables, which are dimensional terms, can be expressed in a dimensionless ratio by expressing q in lineal form by means of the equation for critical depth, $d_c=\sqrt[3]{\frac{q^2}{g}}$, as follows:

$$\frac{d_c}{Y}=\sqrt[3]{\frac{q^2}{gY^3}}$$

From this expression it can be seen that $\frac{q^2}{gY^3}$ is a dimensionless ratio which can be used as an independent variable to which the individual dimensions may be related. This ratio is called the "drop number" and is designated $\overline{D}$. It can be shown that $\overline{D}$ is the product of F_1^2 and $\left(\frac{d_1}{Y}\right)^3$, where F_1 is the Froude number $\frac{v_1}{\sqrt{d_1 g}}$ at the point where the nappe meets the basin floor.

(b) *Hydraulic Jump Basin.*—The jump characteristics of the straight drop basin are basically the same as those for other jump basins, except that the position of the start of the jump cannot be determined as readily as it can for other basins. On figure 219 the point of the start of the jump (point X) will vary with the vertical drop distance and is influenced by the under nappe pool depth, d_f. The basin design downstream from point X will be patterned after those discussed in section 199, once distance L_d is determined. Values of the depth d_1, and of the Froude number, F_1, at the start of the jump in relation to the drop number, $\overline{D}$, are shown on figure 219. These relations may be used for determining the basin dimensions.

Where tailwater depths are greater than the conjugate depth d_2, the jump will move back on the free falling nappe raising the depth d_f of the under nappe pool. With greater depths of the under nappe pool, the nappe will not plunge immediately to the floor of the basin but will be deflected upward along the top of the under pool so that it will meet the floor to the right of point X. The distance to the start of the jump, L_d, will become progressively longer as the tailwater depth is increased. Average values of L_d in relation to $\frac{h_d}{H_e}$, as determined from tests, are plotted on figure 219. For a basin with excessive depths the type II basin discussed in section 199 is most adaptable. The impact block type basin, discussed below, also can be adopted for low drop spillways with excessive tailwater depths.

(c) *Impact Block Type Basin.*—An impact block basin has been developed [1] for low heads which gives reasonably good dissipation of energy for a wide range of tailwater depths. The dissipation of the high energy is principally by turbulence induced by the impingement of the incoming flow upon the impact blocks. The required tailwater depths, therefore, become more or less independent of the drop height. The linear proportions are as follows:

Minimum basin length, $L_B=L_p+2.55\ d_c$

Minimum length to upstream face of baffle block$=L_p+0.8\ d_c$

Minimum tailwater depth, $d_{tw}=2.15\ d_c$

Optimum baffle block height$=0.8\ d_c$

Width and spacing of baffle block$=0.4\ d_c\pm$

Optimum height of end sill$=0.4\ d_c$

(d) *Slotted Grating Dissipator.*—An effective dissipator for small drops is illustrated on figure 220. This device has been tested for values of the Froude number, F_1, as determined at basin apron level, in the range of 2.5 to 4.5. For this arrangement the overfalling sheet is separated into a number of long, thin segments, which fall nearly vertically into the basin below, where dissipation of energy takes place by turbulence. To be effective the length of the grating, L_G, must be such that the entire incoming flow will fall through the slots before reaching the downstream end. The length is therefore a function of the total discharge, the velocity of the incoming flow, and the area of the grating slots. Experimental tests indicate that the following relation gives an effective design:

$$L_G=\frac{Q}{0.245wN\sqrt{2gH_e}} \qquad (27)$$

where:

L_G=the length of the grating in feet,
w =the width of the slot in feet,
N =the number of slots, and
H_e=the depth of flow upstream from the drop.

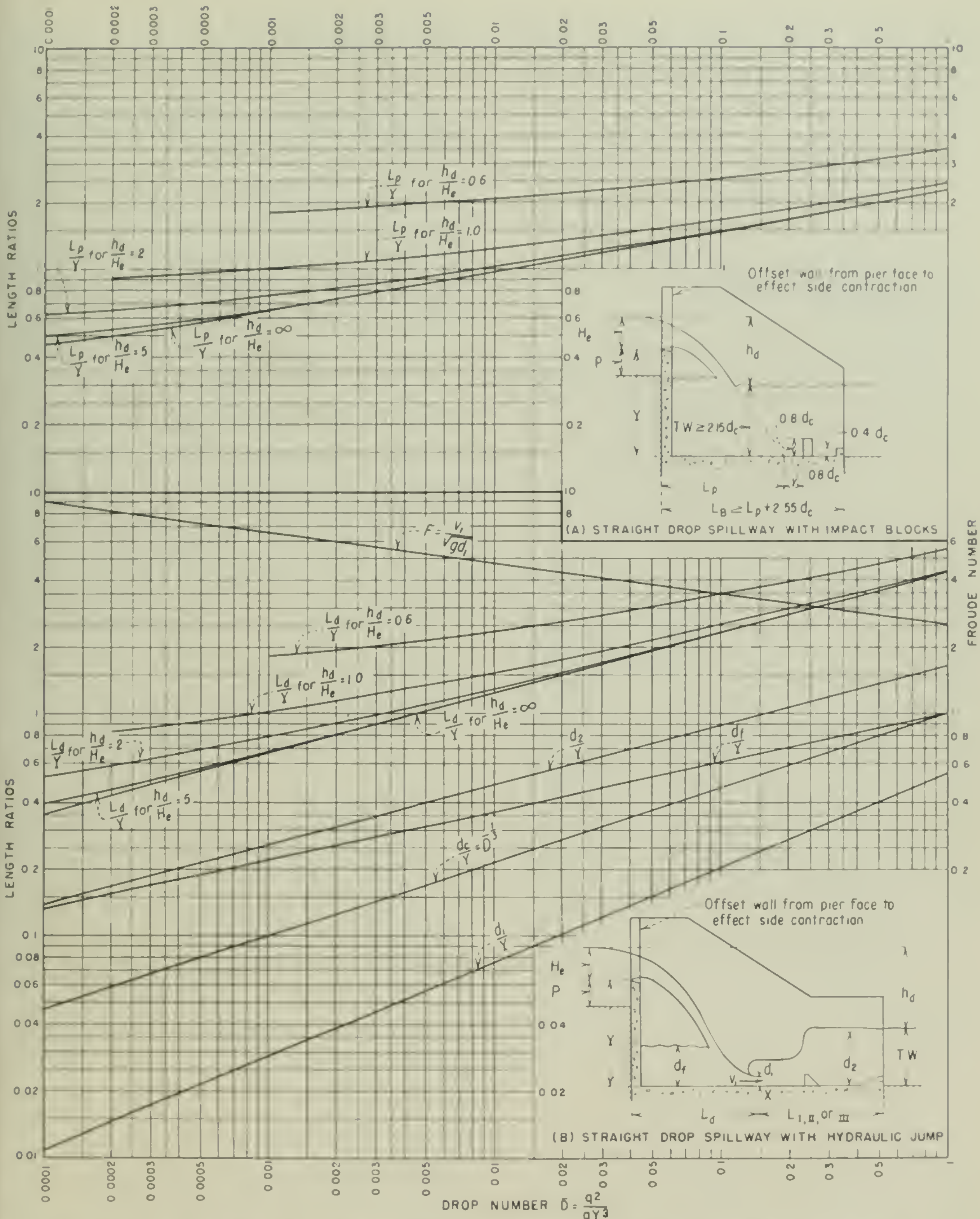


Figure 219. **Hydraulic characteristics of straight drop spillways with hydraulic jump or with impact blocks.**

The length of the basin, L_B, should be approximately 1.2 L_G. An end sill similar to that for basin type I, discussed in section 199, can be provided to improve the hydraulic action.

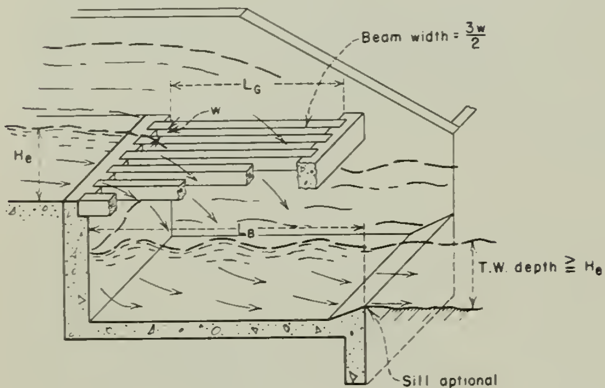


***Figure 220.* Slotted grating dissipator.**

(e) *Example of Design of a Free Overfall Spillway.*—The procedure for designing a free overfall spillway is best shown by means of an example. Consider that such a spillway is to be designed to discharge 500 second-feet. The drop from the spillway crest to the tailwater level for a flow of 500 second-feet is 12 feet. (Tailwater elevation is 108.0.) The approach channel is 20 feet long and the approach floor is level with the spillway crest which is at elevation 120.0. Each type of energy dissipator is to be investigated.

The procedure for design of the *hydraulic jump basin* is as follows: First, assume the effective length of the spillway crest to be 15 feet. Assume an approximate value of $C=3.0$. The unit discharge, q, is equal to $\frac{500}{15}=33.3$ second-feet and H_e is equal to $\left(\frac{q}{C}\right)^{2/3}=\left(\frac{33.3}{3.0}\right)^{2/3}=5.0$ feet. The reservoir water surface elevation, therefore, is $120.0+5.0=125.0$. Thus the drop from reservoir level to tailwater level will be approximately 17 feet.

Assume that an offset of 0.5 foot is provided along each side of the weir to effect side contractions for aerating the underside of the sheet, and that the offset is square-cornered. Then the net crest length, which will also be the stilling basin width, is:

$$L'=L+2K_aH_e+2(0.5)=15+2(0.2)(5)+1.0=18.0 \text{ feet.}$$

Figure 208 is used to determine the approximate apron level of the jump basin, assuming the effective width of the basin to be 15 feet and (for the first trial) that there will be no loss of energy between the reservoir and the point where the jet strikes the basin floor. From scale A, the conjugate depth d_2 for $q=33.3$ second-feet and $H_T=17$ feet is 8.8 feet, which places the apron floor at elevation 99.2. Y is equal to elevation 120 − elevation $99.2=20.8$ feet, and the drop number $\overline{D}$ is equal to $\frac{q^2}{gY^3}=\frac{33.3^2}{32.2\times 20.8^3}=0.0038$. For $\overline{D}=0.0038$, from figure 219 $\frac{d_2}{Y}=0.375$ and $d_2=7.8$ feet. The apron level then must be adjusted to an elevation which is d_2 below the tailwater elevation 108.0, or elevation 100.2.

For the second trial, the adjusted value of Y is 19.8 and $\overline{D}$ is equal to $\frac{33.3^2}{32.2\times 19.8^3}=0.0044$. For $\overline{D}=0.0044$ and $\frac{h_d}{H_e}=\frac{17}{5}=3.4$, from figure 219, $\frac{L_d}{Y}=1.02$ and $L_d=20.2$ feet. Also $d_1=1.1$ feet and $F_1=5.3$.

With the values of $F_1=5.3$, $d_1=1.1$ and $d_2=7.8$, the arrangement of the type II basin shown on figure 206 can be used. From figure 206, $\frac{L_{II}}{d_2}=2.37$ and $L_{II}=18.5$ feet. The length of the basin measured from the vertical crest is equal to $L_d+L_{II}=20.2+18.5=38.7$ feet. The distance of the baffle blocks from the vertical crest for this basin will be 20.2 feet plus 0.8 d_2 or 20.2 plus 0.8 $(7.8)=26.4$ feet, approximately.

The baffle blocks will be approximately 1.5 d_1 or 1.6 feet high and will be about 14 inches wide and spaced at about 28-inch centers.

For the *impact block basin*, the procedure is as follows: The critical depth, d_c, is equal to $\sqrt[3]{\frac{q^2}{g}}=\sqrt[3]{\frac{33.3^2}{32.2}}=3.3$ feet. Then from figure 219, for $\overline{D}=0.0044$ and $\frac{h_d}{H_e}=3.4$, $\frac{L_p}{Y}=0.85$ and $L_p=17.0$ feet. The minimum length of the basin, L_B, is equal to $L_p+2.55\ d_c=17.0+2.55\ (3.3)=25.4$ feet, say 26 feet. The minimum tailwater depth of 2.15 d_c will be 7.1 feet which places the basin

floor at elevation 100.9. The distance from the vertical crest to the baffle blocks will be $L+0.8$ $d_c=17.0+0.8\times3.3=19.6$ feet, say 20 feet. The baffle blocks will be about 0.8 d_c or 3.0 feet high and about 18 inches wide, spaced at about 3-foot centers. The end sill will be 0.4 d_c or about 1.5 feet high.

It can be seen from the above result that if the impact block basin is used, the basin can be made almost 13 feet shorter than that required for a hydraulic jump basin, and also that the impact block basin will be 0.7 foot shallower. The baffle blocks for the hydraulic jump basin will be smaller and spaced closer together than those for the impact block basin.

This example shows that the impact block basin is considerably smaller than the hydraulic jump basin. However, the impact block basin should be limited to use where the drop distance does not exceed 20 feet. Furthermore, as previously explained, the foundation for an impact block basin must be of better quality because of the concentrated forces involved. The hydraulic jump basin, therefore, has a much wider application of use.

The *slotted grating dissipator* is not suitable in this case because the Froude number of 5.3 is in excess of the 4.5 value, which is the tested limit for a practical slotted grating design.

205. *Drop Inlet (Shaft or Morning Glory) Spillways.*—(a) *General Characteristics.*—Typical flow conditions and discharge characteristics of a drop inlet spillway are represented on figure 221. As illustrated on the discharge curve, crest control (condition 1) will prevail for heads between the ordinates of a and g; orifice or tube control (condition 2) will govern for heads between the ordinates of g and h; and the spillway conduit will flow full for heads above the ordinate of h (represented as condition 3).

Flow characteristics of a drop inlet spillway will vary according to the proportional sizes of the different elements. Changing the diameter of the crest will change the curve ab on figure 221 so that the ordinate of g on curve cd will be either higher or lower. For a larger diameter crest, greater outflows can be discharged over the weir at low heads and the transition will fill up and tube control will occur with a lesser head on the crest. Similarly, by altering the size of the throat of the tube, the position of curve cd will change, indicating the heads above which tube control will prevail. If the transition is made of such size that curve cd is moved to coincide with or lie to the right of point j, the control will shift directly from the crest to the downstream end of the conduit. The details of the hydraulic flow characteristics are discussed in following subsections.

(b) *Crest Discharge.*—For small heads, flow over the drop inlet spillway is governed by the characteristics of crest discharge. The vertical transition beyond the crest will flow partly full and the flow will cling to the sides of the shaft. As the discharge over the crest increases, the overflowing annular nappe will become thicker, and eventually the nappe flow will converge into a solid vertical jet. The point where the annular nappe joins the solid jet is called the crotch. After the solid jet forms, a "boil" will occupy the region above the crotch; both the crotch and the top of the boil become progressively higher with larger discharges. For high heads the crotch and boil may almost flood out, showing only a slight depression and eddy at the surface.

Until such time as the nappe converges to form a solid jet, free-discharging weir flow prevails. After the crotch and boil form, submergence begins to affect the weir flow and ultimately the crest will drown out. Flow is then governed either by the nature of the contracted jet which is formed by the overflow entrance, or by the shape and size of the vertical transition if it does not conform to the jet shape. Vortex action must be minimized to maintain converging flow into the drop inlet. Guide piers are often employed along the crest for this purpose [5, 6, 22].

If the crest profile and transition conform to the shape of the lower nappe of a jet flowing over a sharp-crested circular weir, the discharge characteristics for flow over the crest and through the transition can be expressed as:

$$Q=CLH^{3/2} \tag{3}$$

where H is the head measured either to the apex of the under nappe of the overflow, to the spring point of the circular sharp-crested weir, or to some other established point on the overflow. Similarly,

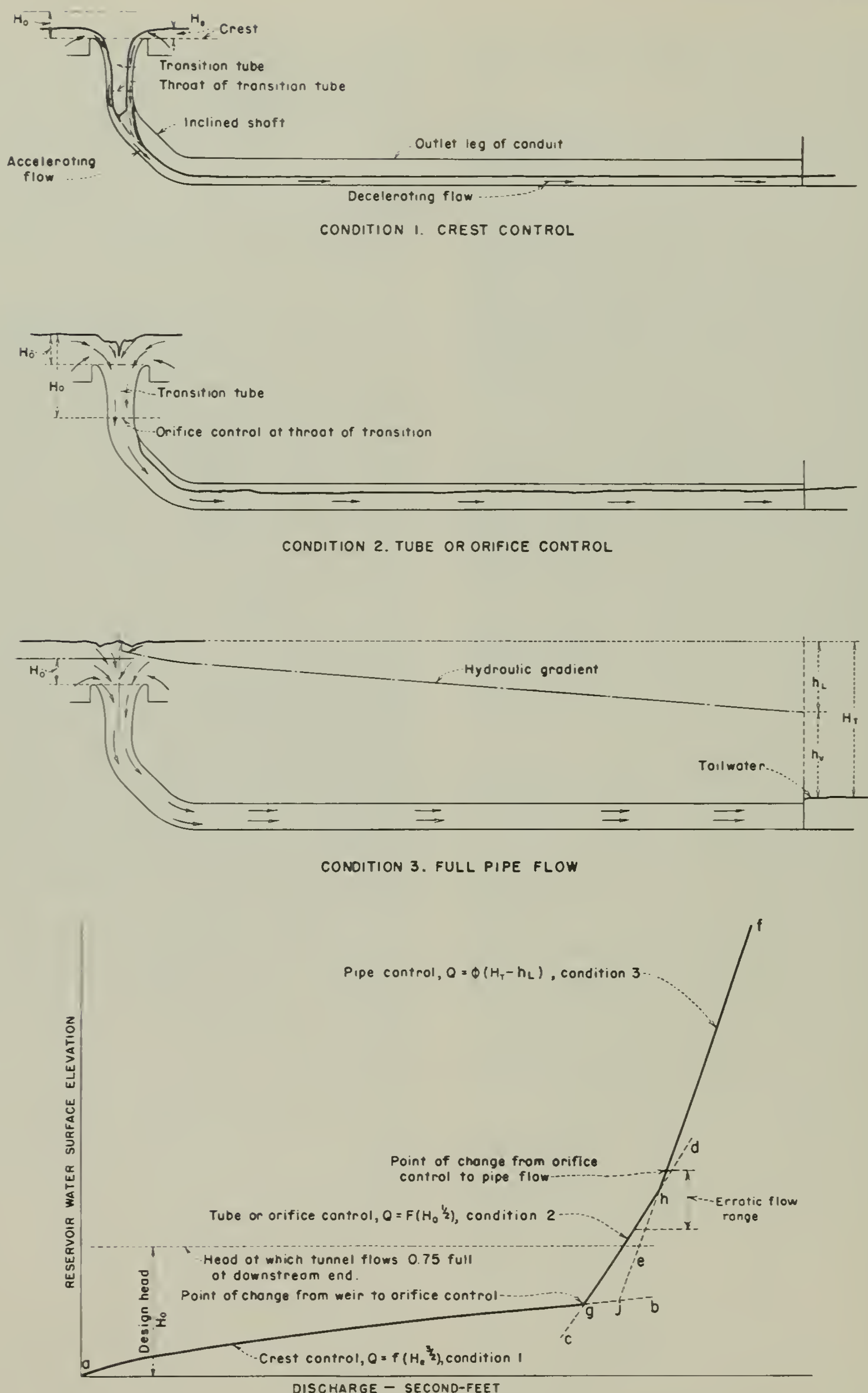


Figure 221. Nature of flow and discharge characteristics of a morning glory spillway.

the choice of the length L is related to some specific point of measurement such as the length of the circle at the apex, along the periphery at the upstream face of the crest, or along some other chosen reference line. The value of C will change with different definitions of L and H. If L is taken at the outside periphery of the overflow crest (the origin of the coordinates in figure 222) and if the head is measured to the apex of the overflow shape, equation (3) can be written:

$$Q = C_o(2\pi R_s)H_o^{3/2} \tag{28}$$

It will be apparent that the coefficient of discharge for a circular crest differs from that for a straight crest because of the effects of submergence and back pressure incident to the joining of the converging flows. Thus C_o must be related to both H_o and R_s, and can be expressed in terms of $\frac{H_o}{R_s}$. The relationship of C_o, as determined from model tests [15], to values of $\frac{H_o}{R_s}$ for three conditions of approach depth is plotted on figure 223. These coefficients are valid only if the crest profile and transition shape conform to that of the jet flowing over a sharp-crested circular weir at H_o head and if aeration is provided so that subatmospheric pressures do not exist along the lower nappe surface contact.

When the crest outline and transition shape conform to the profile of the nappe shape for an H_o head over the crest, free flow prevails for $\frac{H_o}{R_s}$ ratios up to approximately 0.45, and weir control governs. As the $\frac{H_o}{R_s}$ ratio increases above 0.45, the weir partly submerges and flow showing characteristics of a submerged weir is the controlling condition. When the $\frac{H_o}{R_s}$ ratio approaches 1.0, the water surface above the weir is completely submerged. For this and higher stages of $\frac{H_o}{R_s}$, the flow phenomena is that of orifice flow. The weir formula, $Q=CLH^{3/2}$, is used as the measure of flow through the drop inlet entrance regardless of the submergence, by using a coefficient which reflects the flow conditions through the various $\frac{H}{R_s}$ ranges. Thus, from figure 223 it will be seen that the weir coefficient is only slightly changed from that normally indicated for values of $\frac{H_o}{R_s}$ less than 0.45, but reduces rapidly for the higher $\frac{H_o}{R_s}$ ratios.

It will be noted that for most conditions of flow over a circular weir the coefficient of discharge increases with a reduction of the approach depth, whereas the opposite is true for a straight weir. For both weirs a shallower approach lessens the upward vertical velocity component and consequently suppresses the contraction of the nappe. However, for the circular weir the submergence effect is reduced because of a depressed upper nappe surface, giving the jet a quicker downward impetus, which lowers the position of the crotch and increases the discharge.

Coefficients for partial heads of H_e on the crest can be determined from figure 224 to prepare a discharge-head relationship. The designer must be cautious in applying the above criteria, since subatmospheric pressure or submergence effects may alter the flow conditions differently for variously shaped profiles. This criteria, therefore, should not be applied for flow conditions where $\frac{H_e}{R_s}$ exceeds 0.4.

(e) *Crest Profiles.*—Values of coordinates to define the shape of the lower surface of a nappe flowing over an aerated sharp-crested circular weir for various conditions of $\frac{P}{R_s}$ and $\frac{H_s}{R_s}$ are shown

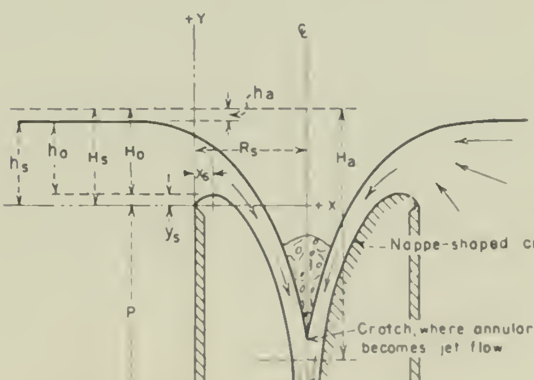


***Figure 222.* Elements of nappe-shaped profile for circular weir.**

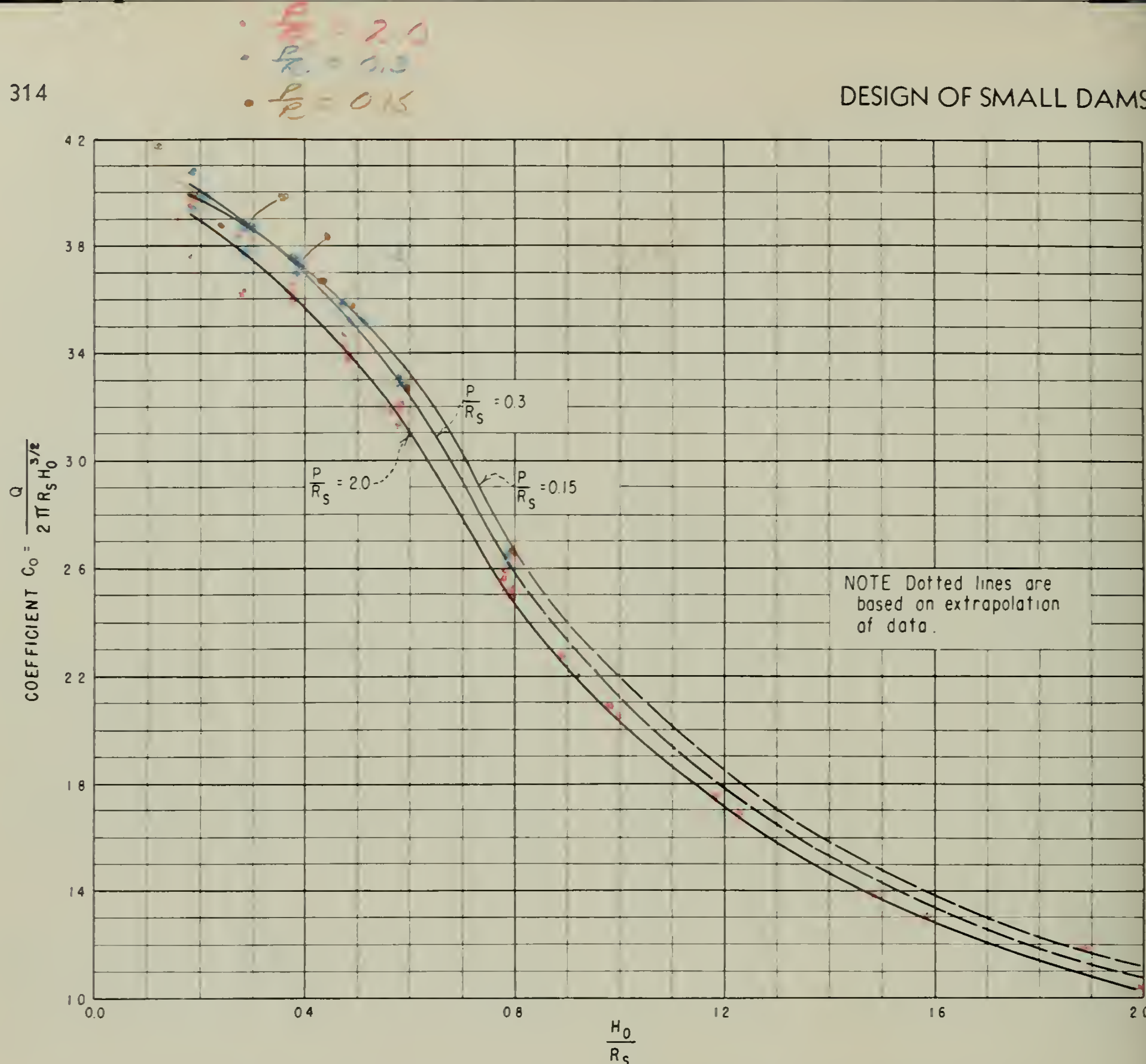


Figure 223. **Relationship of circular crest coefficient C_o to $\frac{H_o}{R_s}$ for different approach depths (aerated nappe).**

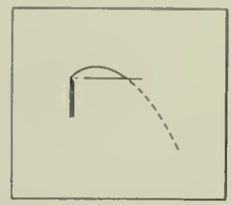
in tables 22, 23, and 24. These data are based on experimental tests [15] conducted by the Bureau of Reclamation. The relationships of H_s to H_o are shown on figure 225. Typical upper and lower nappe profiles for various values of $\frac{H_s}{R_s}$ are plotted on figure 226 in terms of $\frac{x}{H_s}$ and $\frac{y}{H_s}$ for the condition of $\frac{P}{R_s}=2.0$.

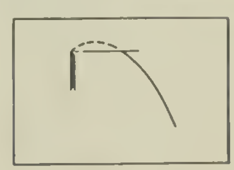
Illustrated on figure 227 are typical lower nappe profiles, plotted for various values of H_s for a given value of R_s. In contrast to the straight weir where the nappe springs farther from the crest as the head increases, it will be seen from figure 227 that the lower nappe profile for the circular crest springs farther only in the region of the high point of the trace, and then only for $\frac{H_s}{R_s}$ values up to about 0.5. The profiles become increasingly suppressed for larger $\frac{H_s}{R_s}$ values. Below the high point of the profile the traces cross and the shapes for the higher heads fall inside those for the lower heads. Thus, if the crest profile is designed for heads where $\frac{H_s}{R_s}$ exceeds about 0.25 to 0.3, it appears that subatmospheric pressure will occur along some portion of the profile when heads are less than the designed maximum. If subatmospheric pressures are to be avoided along the crest profile, the crest shape should be selected so that it will give support

TABLE 22.—*Coordinates of lower nappe surface for different values of* $\frac{H_s}{R}$ *when* $\frac{P}{R}=2$

[Negligible approach velocity and aerated nappe]

$\frac{H_s}{R}$	0.00	0.10*	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.60	0.80	1.00	1.20	1.50	2.00
$\frac{X}{H_s}$	$\frac{Y}{H_s}$ For portion of the profile above the weir crest														
0.000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
.010	.0150	.0145	0133	.0130	.0128	.0125	.0122	.0119	.0116	.0112	0104	.0095	0086	0077	0070
.020	.0280	0265	0250	0243	.0236	.0231	.0225	.0220	.0213	0202	.0180	.0159	0140	0115	[illegible]
.030	.0395	.0365	.0350	.0337	.0327	.0317	.0308	.0299	.0289	.0270	.0231	.0198	.0168	0126	0085
.040	.0490	.0460	.0435	.0417	.0403	.0389	.0377	.0363	.0351	.0324	.0268	.0220	0176	0117	0050
.050	.0575	.0535	.0506	.0487	0471	.0454	.0436	.0420	.0402	.0368	.0292	.0226	0168	0092	
.060	.0650	.0605	.0570	0550	.0531	.0510	.0489	.0470	.0448	.0404	.0305	.0220	.0147	0053	
.070	.0710	0665	.0627	0605	0584	.0560	.0537	0514	.0487	.0432	0308	.0201	.0114	0001	
.080	.0765	.0710	0677	.0655	.0630	.0603	.0578	.0550	.0521	.0455	0301	.0172	0070		
.090	.0820	.0765	0722	.0696	.0670	.0640	.0613	.0581	.0549	.0471	.0287	.0135	.0018		
.100	.0860	.0810	.0762	.0734	.0705	.0672	.0642	.0606	.0570	.0482	.0264	.0089			
.120	0940	.0880	.0826	.0790	.0758	.0720	.0683	.0640	.0596	.0483	.0195				
.140	.1000	.0935	.0872	.0829	.0792	.0750	.0705	.0654	.0599	.0460	.0101				
.160	.1045	.0980	.0905	.0855	.0812	.0765	.0710	.0651	.0585	.0418					
.180	1080	.1010	.0927	.0872	.0820	0766	0705	.0637	.0559	0361					
.200	.1105	1025	.0938	.0877	.0819	.0756	.0688	0611	.0521	0292					
.250	.1120	1035	.0926	.0850	.0773	.0683	.0596	.0495	.0380	0068					
.300	.1105	1000	.0850	.0764	.0668	.0559	.0446	.0327	.0174						
.350	.1060	.0930	.0750	.0650	.0540	.0410	.0280	.0125							
.400	.0970	.0830	.0620	.0500	.0365	.0220	.0060								
.450	.0845	.0700	.0450	.0310	.0170	.000									
.500	.0700	.0520	.0250	.0100											
.550	.0520	.0320	0020												
.600	.0320	0080													
.650	.0090														
$\frac{Y}{H_s}$	$\frac{X}{H_s}$ For portion of the profile below the weir crest														
0.000	0.668	0.615	0.554	0.520	0.487	0.450	0.413	0.376	0.334	0.262	0.158	0.116	0.093	0.070	0.048
−.020	705	.652	.592	.560	.526	.488	.452	.414	.369	.293	.185	.145	.120	.096	.074
−.040	.742	688	.627	.596	.563	.524	.487	.448	.400	.320	.212	.165	.140	.115	.088
−.060	.777	.720	.660	.630	.596	.557	.519	.478	.428	.342	.232	.182	.155	.129	.100
−.080	.808	.752	692	.662	.628	.589	.549	.506	.454	.363	.250	.197	.169	.140	.110
−.100	.838	.784	.722	.692	.657	.618	.577	.532	.478	.381	.266	.210	.180	.150	.118
−.150	.913	.857	.793	.762	.725	.686	.641	.589	.531	.423	.299	.238	.204	.170	.132
−.200	.978	.925	.860	.826	.790	.745	.698	.640	.575	.459	.326	.260	.224	.181	.144
−.250	1.040	985	.919	.883	.847	.801	.750	.683	.613	.490	.348	.280	.239	.193	.153
−.300	1.100	1.043	.976	.941	.900	.852	.797	.722	.648	.518	.368	.296	.251	.206	.160
−.400	1.207	1.150	1.079	1.041	1.000	.944	.880	.791	.706	.562	.400	.322	.271	.220	168
−.500	1.308	1.246	1.172	1.131	1.087	1.027	.951	.849	.753	.598	.427	.342	.287	.232	.173
−.600	1.397	1.335	1.260	1.215	1.167	1.102	1.012	.898	.793	.627	.449	.359	.300	240	.179
−.800	1.563	1.500	1.422	1.369	1.312	1.231	1.112	.974	.854	.673	.482	.384	.320	.253	.184
−1.000	1.713	1.646	1.564	1.508	1.440	1.337	1.189	1.030	.899	.710	.508	.402	.332	260	188
−1.200	1.846	1.780	1.691	1.635	1.553	1.422	1.248	1.074	.933	.739	.528	.417	.340	266	
−1.400	1.970	1.903	1.808	1.748	1.653	1.492	1.293	1.108	.963	.760	.542	.423	.344		
−1.600	2.085	2.020	1.918	1.855	1.742	1.548	1.330	1.133	.988	.780	.553	.430			
−1.800	2.196	2.130	2.024	1.957	1.821	1.591	1.358	1.158	1.008	.797	.563	.433			
−2.000	2.302	2.234	2.126	2.053	1.891	1.630	1.381	1.180	1.025	.810	.572				
−2.500	2.557	2.475	2.354	2.266	2.027	1.701	1.430	1.221	1.059	.838	.588				
−3.000	2.778	2.700	2.559	2.428	2.119	1.748	1.468	1.252	1.086	.853					
−3.500	.	2.916	2.749	2.541	2.171	1.777	1.489	1.267	1.102						
−4.000	.	3.114	2.914	2.620	2.201	1.796	1.500	1.280							
−4.500	.	3.306	3.053	2.682	2.220	1.806	1.509								
−5.000	.	3.488	3.178	2.734	2.227	1.811									
−5.500	.	3.653	3.294	2.779	2.229										
−6.000	.	3.820	3.405	2.812	2.232										
$\frac{H_s}{R}$	0.00	0.10	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.60	0.80	1.00	1.20	1.50	2.00





*The tabulation for $\frac{H_s}{R}=0.10$ was obtained by interpolation between $\frac{H_s}{R}=0$ and 0.20.

After Wagner [15]

TABLE 23.—*Coordinates of lower nappe surface for different values of* $\frac{H_s}{R}$ *when* $\frac{P}{R}=0.30$

$\frac{H_s}{R}$	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.60	0.80
$\frac{X}{H_s}$	$\frac{Y}{H_s}$ For portion of the profile above the weir crest								
0.000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
.010	.0130	.0130	.0130	.0125	.0120	.0120	.0115	.0110	.0100
.020	.0245	.0242	.0240	.0235	.0225	.0210	.0195	.0180	.0170
.030	.0340	.0335	.0330	.0320	.0300	.0290	.0270	.0240	.0210
.040	.0415	.0411	.0390	.0380	.0365	.0350	.0320	.0285	.0240
.050	.0495	.0470	.0455	.0440	.0420	.0395	.0370	.0325	.0245
.060	.0560	.0530	.0505	.0490	.0460	.0440	.0405	.0350	.0250
.070	.0610	.0575	.0550	.0530	.0500	.0470	.0440	.0370	.0245
.080	.0660	.0620	.0590	.0565	.0530	.0500	.0460	.0385	.0235
.090	.0705	.0660	.0625	.0595	.0550	.0520	.0480	.0390	.0215
.100	.0740	.0690	.0660	.0620	.0575	.0540	.0500	.0395	.0190
.120	.0800	.0750	.0705	.0650	.0600	.0560	.0510	.0380	.0120
.140	.0840	.0790	.0735	.0670	.0615	.0560	.0515	.0355	.0020
.160	.0870	.0810	.0750	.0675	.0610	.0550	.0500	.0310	
.180	.0885	.0820	.0755	.0675	.0600	.0535	.0475	.0250	
.200	.0885	.0820	.0745	.0660	.0575	.0505	.0435	.0180	
.250	.0855	.0765	.0685	.0590	.0480	.0390	.0270		
.300	.0780	.0670	.0580	.0460	.0340	.0220	.0050		
.350	.0660	.0540	.0425	.0295	.0150				
.400	.0495	.0370	.0240	.0100					
.450	.0300	.0170	.0025						
.500	.0090	−.0060							
.550									
$\frac{Y}{H_s}$	$\frac{X}{H_s}$ For portion of the profile below the weir crest								
−0.000	0.519	0.488	0.455	0.422	0.384	0.349	0.310	0.238	0.144
−.020	.560	.528	.495	.462	.423	.387	.345	.272	.174
−.040	.598	.566	.532	.498	.458	.420	.376	.300	.198
−.060	.632	.601	.567	.532	.491	.451	.406	.324	.220
−.080	.664	.634	.600	.564	.522	.480	.432	.348	.238
−.100	.693	.664	.631	.594	.552	.508	.456	.368	.254
−.150	.760	.734	.701	.661	.618	.569	.510	.412	.290
−.200	.831	.799	.763	.723	.677	.622	.558	.451	.317
−.250	.893	.860	.826	.781	.729	.667	.599	.483	.341
−.300	.953	.918	.880	.832	.779	.708	.634	.510	.362
−.400	1.060	1.024	.981	.932	.867	.780	.692	.556	.396
−.500	1.156	1.119	1.072	1.020	.938	.841	.745	.595	.424
−.600	1.242	1.203	1.153	1.098	1.000	.891	.780	.627	.446
−.800	1.403	1.359	1.301	1.227	1.101	.970	.845	.672	.478
−1.000	1.549	1.498	1.430	1.333	1.180	1.028	.892	.707	.504
−1.200	1.680	1.622	1.543	1.419	1.240	1.070	.930	.733	.524
−1.400	1.800	1.739	1.647	1.489	1.287	1.106	.959	.757	.540
−1.600	1.912	1.849	1.740	1.546	1.323	1.131	.983	.778	.551
−1.800	2.018	1.951	1.821	1.590	1.353	1.155	1.005	.797	.560
−2.000	2.120	2.049	1.892	1.627	1.380	1.175	1.022	.810	.569
−2.500	2.351	2.261	2.027	1.697	1.428	1.218	1.059	.837	
−3.000	2.557	2.423	2.113	1.747	1.464	1.247	1.081	.852	
−3.500	2.748	2.536	2.167	1.778	1.489	1.263	1.099		
−4.000	2.911	2.617	2.200	1.796	1.499	1.274			
−4.500	3.052	2.677	2.217	1.805	1.507				
−5.000	3.173	2.731	2.223	1.810					
−5.500	3.290	2.773	2.228						
−6.000	3.400	2.808							
$\frac{H_s}{R}$	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.60	0.80

After Wagner [15]

TABLE 24. *Coordinates of lower nappe surface for different values of* $\frac{H_s}{R}$ *when* $\frac{P}{R} = 0.15$

$\frac{H_s}{R}$	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.60	0.80
$\frac{X}{H_s}$	$\frac{Y}{H_s}$ For portion of the profile above the weir crest								
0.000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
.010	.0120	.0120	.0115	.0115	.0110	.0110	.0105	.0100	.0090
.020	.0210	.0200	.0195	.0190	.0185	.0180	.0170	.0160	.0140
.030	.0285	.0270	.0265	.0260	.0250	.0235	.0225	.0200	.0165
.040	.0345	.0335	.0325	.0310	.0300	.0285	.0265	.0230	.0170
.050	.0405	.0385	.0375	.0360	.0345	.0320	.0300	.0250	.0170
.060	.0450	.0430	.0420	.0400	.0380	.0355	.0330	.0265	.0165
.070	.0495	.0470	.0455	.0430	.0410	.0380	.0350	.0270	.0150
.080	.0525	.0500	.0485	.0460	.0435	.0400	.0365	.0270	.0130
.090	.0560	.0530	.0510	.0480	.0455	.0420	.0370	.0265	.0100
.100	.0590	.0560	.0535	.0500	.0465	.0425	.0375	.0255	.0065
.120	.0630	.0600	.0570	.0520	.0480	.0435	.0365	.0220	
.140	.0660	.0620	.0585	.0525	.0475	.0425	.0345	.0175	
.160	.0670	.0635	.0590	.0520	.0460	.0400	.0305	.0110	
.180	.0675	.0635	.0580	.0500	.0435	.0365	.0260	.0040	
.200	.0670	.0625	.0560	.0465	.0395	.0320	.0200		
.250	.0615	.0560	.0470	.0360	.0265	.0160	.0015		
.300	.0520	.0410	.0330	.0210	.0100				
.350	.0380	.0285	.0165	.0030					
.400	.0210	.0090							
.450	.0015								
.500									
.550									
$\frac{Y}{H_s}$	$\frac{X}{H_s}$ For portion of the profile below the weir crest								
−0.000	0.454	0.422	0.392	0.358	0.325	0.288	0.253	0.189	0.116
−.020	.499	.467	.437	.404	.369	.330	.292	.228	.149
−.040	.540	.509	.478	.444	.407	.368	.328	.259	.174
−.060	.579	.547	.516	.482	.443	.402	.358	.286	.195
−.080	.615	.583	.550	.516	.476	.434	.386	.310	.213
−.100	.650	.616	.584	.547	.506	.462	.412	.331	.228
−.150	.726	.691	.660	.620	.577	.526	.468	.376	.263
−.200	.795	.760	.729	.685	.639	.580	.516	.413	.293
−.250	.862	.827	.790	.743	.692	.627	.557	.445	.319
−.300	.922	.883	.843	.797	.741	.671	.594	.474	.342
−.400	1.029	.988	.947	.893	.828	.749	.656	.523	.381
−.500	1.128	1.086	1.040	.980	.902	.816	.710	.567	.413
−.600	1.220	1.177	1.129	1.061	.967	.869	.753	.601	.439
−.800	1.380	1.337	1.285	1.202	1.080	.953	.827	.655	.473
−1.000	1.525	1.481	1.420	1.317	1.164	1.014	.878	.696	.498
−1.200	1.659	1.610	1.537	1.411	1.228	1.059	.917	.725	.517
−1.400	1.780	1.731	1.639	1.480	1.276	1.096	.949	.750	.531
−1.600	1.897	1.843	1.729	1.533	1.316	1.123	.973	.770	.544
−1.800	2.003	1.947	1.809	1.580	1.347	1.147	.997	.787	.553
−2.000	2.104	2.042	1.879	1.619	1.372	1.167	1.013	.801	.560
−2.500	2.340	2.251	2.017	1.690	1.423	1.210	1.049	.827	
−3.000	2.550	2.414	2.105	1.738	1.457	1.240	1.073	.840	
−3.500	2.740	2.530	2.153	1.768	1.475	1.252	1.088		
−4.000	2.904	2.609	2.180	1.780	1.487	1.263			
−4.500	3.048	2.671	2.198	1.790	1.491				
−5.000	3.169	2.727	2.207	1.793					
−5.500	3.286	2.769	2.210						
−6.000	3.396	2.800							
$\frac{H_s}{R}$	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.60	0.80

After Wagner [15]

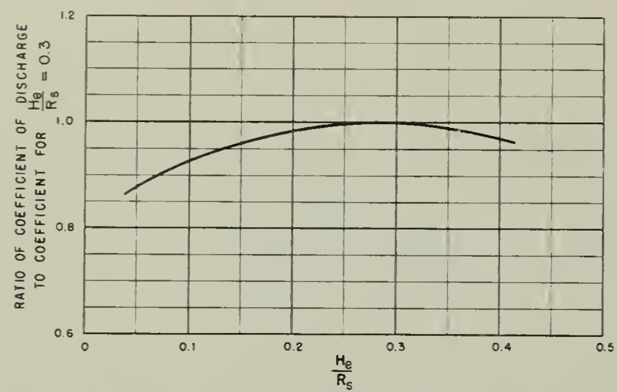


Figure 224. **Circular crest coefficient of discharge for other than design head.**

to the overflow nappe for the smaller $\frac{H_e}{R_s}$ ratios. Figure 228 shows the approximate increase in radius required to minimize subatmospheric pressures on the crest. The crest shape for the enlarged crest radius is then based on a $\frac{H_s'}{R_s'}$ ratio of 0.3.

(d) *Transition Design.*—The diameter of a jet issuing from a horizontal orifice can be determined for any point below the water surface if it is assumed that the continuity equation, $Q=av$, is valid and if friction and other losses are neglected.

For a circular jet the area is equal to πR^2. The discharge is equal to $av=\pi R^2\sqrt{2gh_v}$. Solving for R, $R=\frac{Q_a^{1/2}}{5H_a^{1/4}}$ where H_a is equal to the difference between the water surface and the elevation under consideration. The diameter of the jet thus decreases indefinitely with the distance of the vertical fall.

If an assumed total loss (to allow for jet contraction losses, friction losses, velocity losses due to direction change, etc.) is taken as $0.1H_a$, the equation for determing the approximate required shaft radius can be written:

$$R=0.204\frac{Q^{1/2}}{H_a^{1/4}} \tag{29}$$

Since this equation is for the shape of the jet, its use for determining the shape of the shaft will result in the minimum size which will accommodate the flow without restrictions and without developing pressures along the side of the shaft.

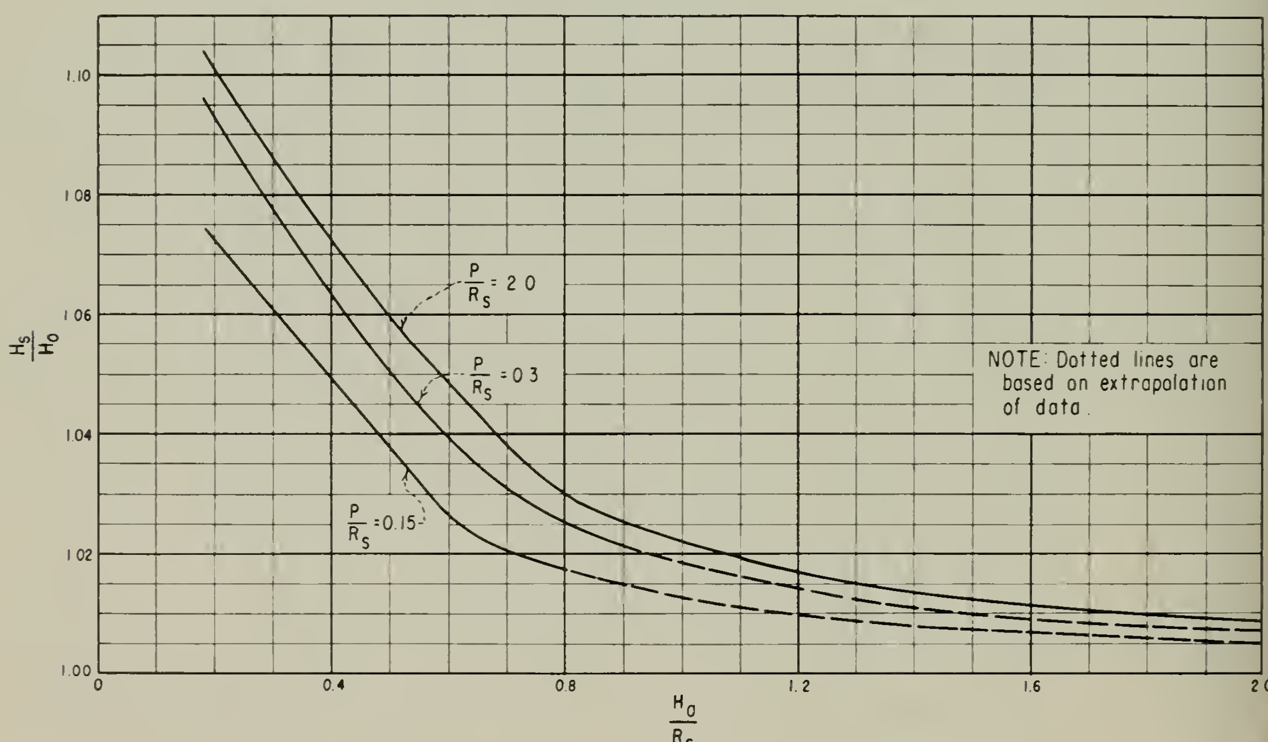


Figure 225. **Relationship of $\frac{H_s}{H_o}$ to $\frac{H_o}{R_s}$ for circular sharp-crested weirs.**

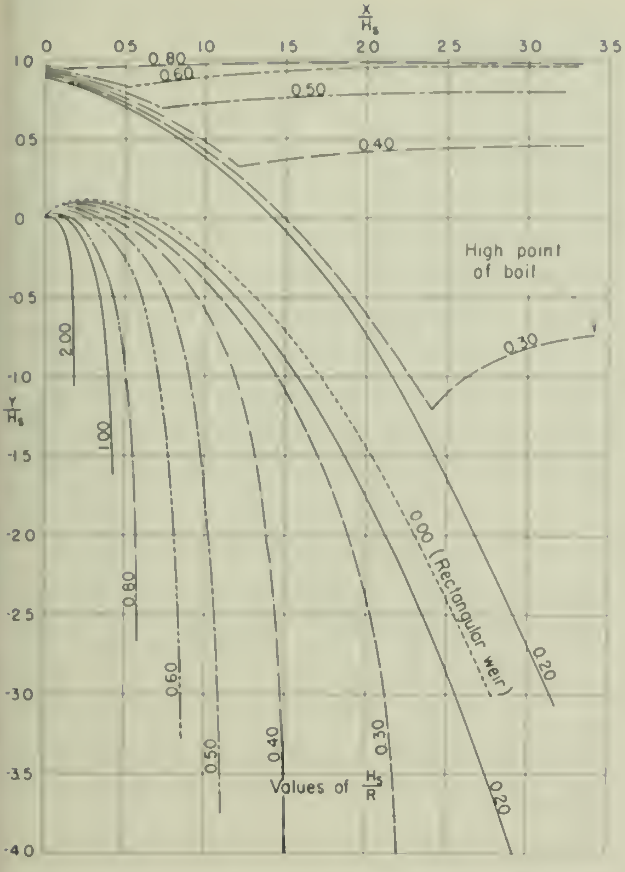


Figure 226. **Upper and lower nappe profiles for circular weir (aerated nappe and negligible approach velocity).**

A typical shaft profile required by equation (29) is shown by the dot-dash lines *abc* on figure 229. If the shaft profile *abc* is enlarged above points *b* as shown by the dotted lines *db*, the section at A–A will act as a control and the shaft above section A–A will flow under pressure; below section A–A the shaft will flow full but it will not be under pressure. If the shaft profile *dbc* is further modified as shown by the solid lines *be*, the shaft will not flow full below section A–A. The length of the shaft below section A–A will not affect the discharge provided the flow is aerated. In this case, neglecting air friction losses, the jet below section A–A will occupy an equivalent area indicated by the lines *bc*. It is interesting to note that with a profile *abc* established for a specific head, the control will remain at section A–A for any higher head, and flow above that section will be under pressure. Conversely, for lower heads, the control point will move upward; above that control

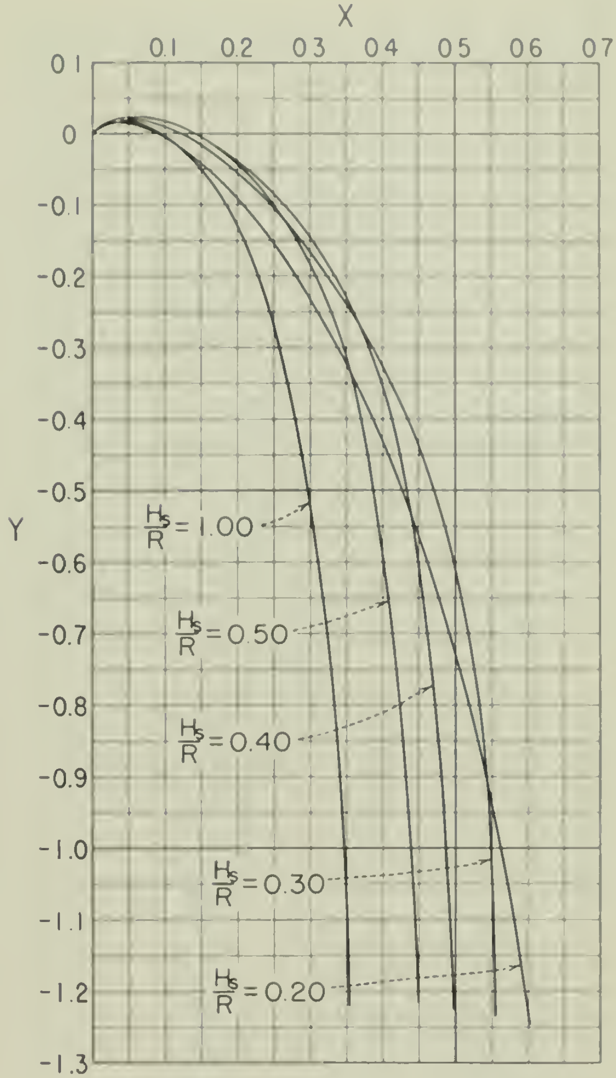


Figure 227. **Comparison of lower nappe shapes for a circular weir for different heads.**

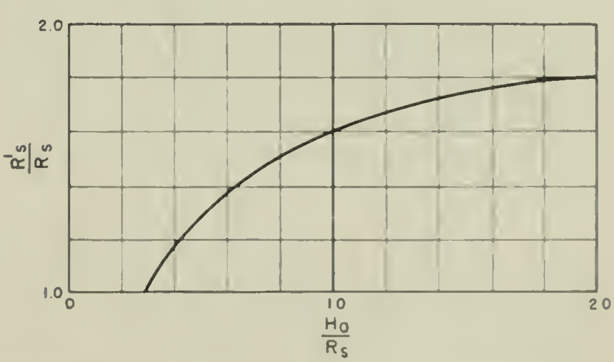


Figure 228. **Increased circular crest radius needed to minimize subatmospheric pressure along crest.**

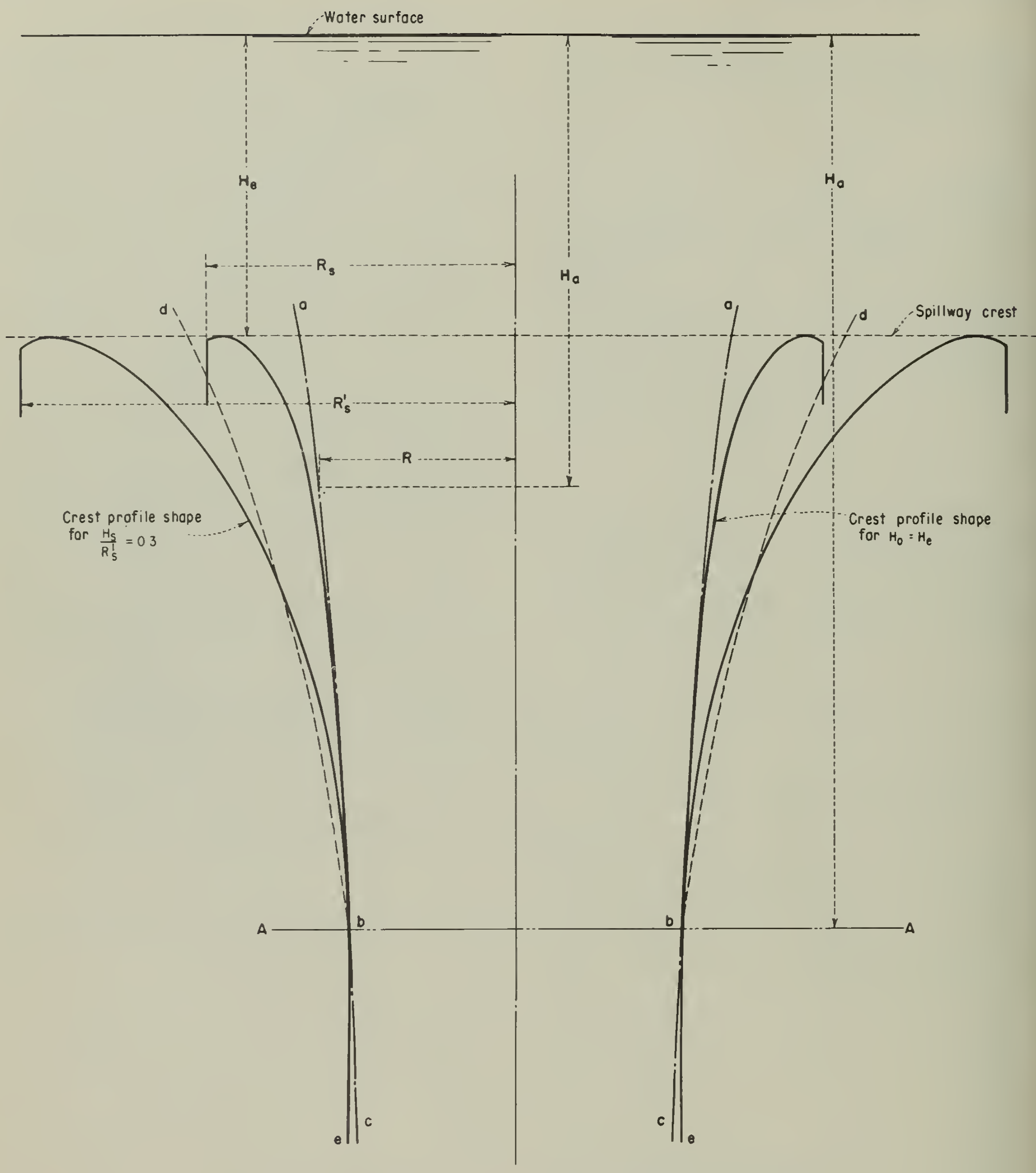


Figure 229. **Comparison of drop inlet profiles for various flow conditions.**

point the shaft will be under pressure, below that control point the shaft will be partly full. If the profile *dbe* is adopted, once a head is reached to make the shaft flow full at point *b*, section A–A will be the point of control, and pressure flow above the control will prevail for that and all greater heads.

For submerged crest flow, the corresponding nappe shape as determined from subsection 205(c) for a design head H_o will be such that along its

lower levels it will closely follow the profile determined from equation (29) if H_e approximates H_o. It must be remembered that on the basis of the losses assumed in equation (29), profile *abc* will be the minimum shaft size which will accommodate the required flow and that no part of the crest shape should be permitted to project inside this profile. As has been noted in section 192, small subatmospheric crest pressures can be tolerated if proper precautions are taken to obtain a smooth surface and if the negative pressure forces are recognized in the structural design. The choice of the minimum crest and transition shapes in preference to some wider shape then becomes a matter of economics, structural arrangement, and layout adaptability.

Where the transition profile corresponds to the continuation of the crest shape as determined by tables 22, 23, and 24, the discharge can be computed from equation (28) using a coefficient from figure 223. Where the transition profile differs from the crest shape profile so that a constricted control section is established, the discharge must be determined from equation (29). On figure 221 the discharge-head relationship curve *ag* will then be computed from the coefficients determined from figure 224 while the discharge-head relationship curve *gh* will be based on equation (29).

(e) *Conduit Design.*—If, for a designated discharge, the conduit of a drop inlet spillway were to flow full below the transition without being under pressure, the required size of the shaft and outlet leg would vary according to the available net head along its length. So long as the slope of the hydraulic gradient which is dictated by the hydraulic losses is flatter than the slope of the conduit, the flow will accelerate and the conduit will decrease in size. When the conduit slope becomes flatter than the slope of the hydraulic gradient, flow will decelerate and the conduit size will increase. All points along the conduit will act simultaneously to control the rate of flow. For heads in excess of that used to proportion the conduit, it will flow under pressure with the control at the downstream end; for heads less than that used to determine the size, the conduit will flow partly full for its entire length and the control will remain in the transition upstream. On figure 221, the head at which the conduit just flows full is represented by point *h*. At heads above point *h* the conduit flows full under pressure; at heads less than *h* the conduit flows partly full with controlling conditions dictated by the transition design.

Because it is impractical to build a conduit with a varying diameter, it is ordinarily made of constant size beyond the inlet transition. Thus, the conduit from the control point in the transition to the downstream end will have an excess of area. If atmospheric pressure can be maintained along the portion of the conduit flowing partly full, it will continue to flow at that stage even though the downstream end fills. Progressively greater discharges will not alter the part full flow in the upper part of the conduit, but full flow conditions under pressure will occupy increasing lengths of the downstream end of the conduit. At the discharge represented by point *h* on figure 221, the full flow condition has moved back to the transition control section and the conduit will flow full for its entire length.

If the conduit flows at such a stage that the downstream end flows full, both the inlet and outlet will be sealed. To forestall siphon action by the withdrawal of air from the conduit would require an adequate venting system. Unless venting is effected over the entire length of conduit, it may prove inadequate to prevent subatmospheric pressures along some portion of the length, because of the possibility of sealing at any point by surging, wave action, or eddy turbulences. Thus, if no venting is provided or if the venting is inadequate, a make-and-break siphon action will attend the flow in the range of discharges approaching full flow conditions. This action is accompanied by erratic discharges, by thumping and vibrations, and by surges at the entrance and outlet of the spillway.

To avoid the possibility of siphonic flow conditions, the downstream conduit size for ordinary designs (and especially for those for higher heads) is chosen so that it will never flow full beyond the inlet transition. To allow for air bulking, surging, etc., the conduit size is selected so that its area will not flow more than 75 percent full at the downstream end at maximum discharges. Under this limitation, air ordinarily will be able to pass up the conduit from the downstream portal and thus prevent the formation of subatmospheric pressure along the conduit length. Precautions must be taken, however, in selecting vertical or horizontal curvature of the conduit profile and

alinement to prevent sealing along some portion by surging or wave action.

(f) *Design Example.*—The following example illustrates the procedure for designing a morning-glory type of drop inlet spillway. Consider that an ungated drop inlet spillway which is to operate under a maximum surcharge head of 10 feet, but which must limit the outflow to 2,000 second-feet, is to be designed. Determine alternate crest shapes and discharge head relationships, considering (1) that the crest radius must be minimized because the intake is formed as a tower away from the abutment and that subatmospheric pressures along the overflow crest can be tolerated, and (2) that the crest can be of any size because it is located on a knoll at the abutment and that it is desired to keep subatmospheric pressures along the crest to a minimum. In either case the conduit must not flow more than 75 percent full at the downstream end. The controlling dimensions are as shown on figure 230.

Case 1.—Radius of crest to be minimized and subatmospheric pressures may be tolerated.

Assume $\frac{P}{R_s} \geq 2$ (see fig. 222). The value of R_s is determined by means of a trial and error procedure of assuming values of R_s and computing the discharge.

Assume $R_s=7.0$ feet. $\frac{H_o}{R_s}=\frac{10}{7}=1.43$. For $\frac{H_o}{R_s}=1.43$ and $\frac{P}{R_s} \geq 2$, from figure 223, $C_o=1.44$. Then $Q=C_o(2\pi R_s)H_o^{3/2}=1.44\ (2\pi)\ (7.0)\ (10)^{3/2}=2{,}010$ second-feet, which is approximately the required discharge. From figure 225 for $\frac{H_o}{R_s}=1.43$ and $\frac{P}{R_s} \geq 2$, $\frac{H_s}{H_o}=1.014$; H_s is equal to $1.014H_o=1.014(10)=10.14$ feet. Then $\frac{H_s}{R_s}$ is equal to $\frac{10.14}{7.0}=1.45$.

Using table 22, points on the profile of the crest shape which conforms to the lower nappe surface for an $\frac{H_s}{R_s}=1.45$ are computed by interpolation and plotted as shown on figure 231.

The next step is to determine the transition shape which will be required to pass 2,000 second-feet with an H_o of 10 feet above the crest (water surface elevation 110). This shape is determined by the use of equation (29):

$$R=0.204\ \frac{Q^{1/2}}{H_a^{1/4}}=0.204\ \frac{(2{,}000)^{1/2}}{H_a^{1/4}}=\frac{9.12}{H_a^{1/4}}$$

Points on the transition are computed as shown in the following table and plotted on the same sheet on which points for the crest shape have already been plotted (fig. 231).

Elevation of section	H_a	$H_a^{1/4}$	$R=\frac{9.12}{H_a^{1/4}}$
100	10	1.78	5.13
98	12	1.86	4.90
96	14	1.93	4.72
94	16	2.00	4.56
92	18	2.06	4.43
88	22	2.17	4.20

A smooth curve is drawn through the controlling points on the crest and transition shapes, determining the final shape of the crest and transition.

The last step is to determine the minimum uniform conduit diameter which will pass the flow from the transition section to the conduit portal without the conduit flowing more than 75 percent full. The procedure is as follows: (1) Select a trial conduit and throat diameter and find the

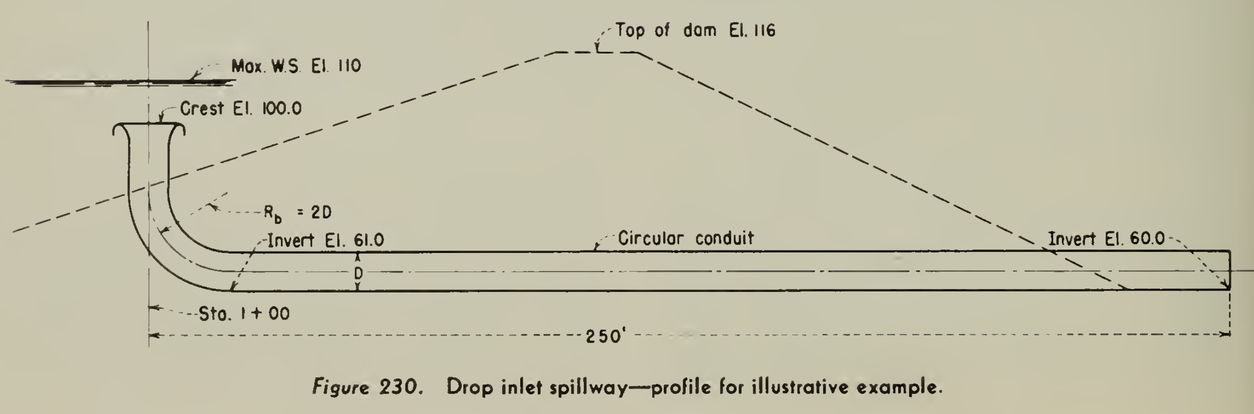


Figure 230. **Drop inlet spillway—profile for illustrative example.**

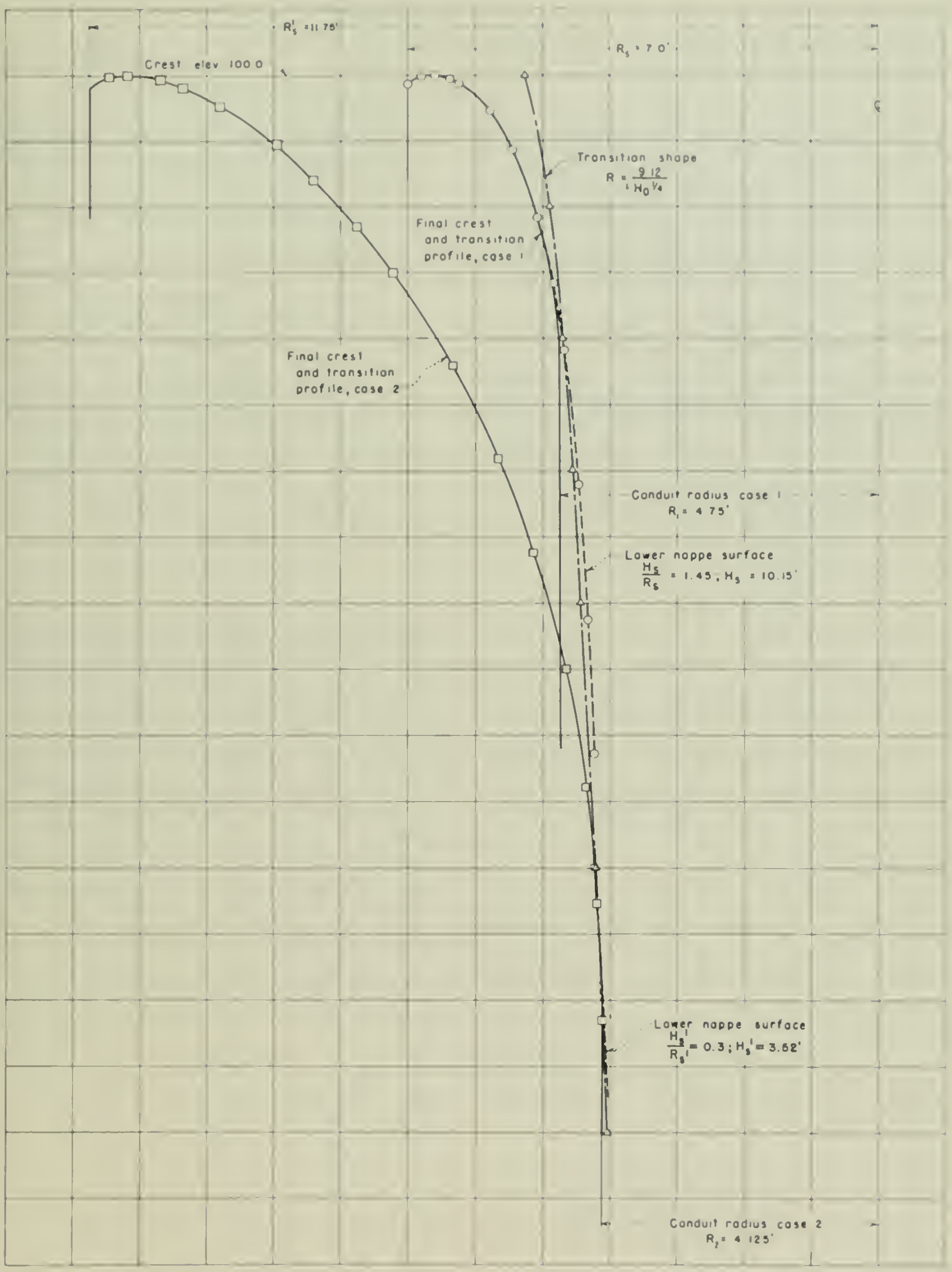


Figure 231. Drop inlet crest, transition, and conduit plottings for illustrative example.

corresponding throat location, (2) compute the length from transition throat to outlet portal, (3) approximate the friction losses in the conduit by assuming the conduit flows three-fourths full for its entire length, and (4) check the elevation of the invert at the outlet portal required to pass the design discharge through the selected size conduit. After an approximate conduit size has been determined in this manner, it should be checked by computing the water surface profile through the conduit by means of open channel flow computations.

For this problem assume a conduit diameter of 9.0 feet. From figure 231, a radius of 4.5 feet is found to be at 6.9 feet below the crest. Thus the elevation of the 9.0-foot-diameter throat is 93.1. The tunnel length may be scaled or calculated by approximate methods. In this example the approximate tunnel length is 270 feet. Assuming that the conduit flows 75 percent full,

$$\text{Area}=0.75(\pi)(4.5^2)=47.7 \text{ square feet}$$

$$\text{Velocity}=\frac{2{,}000}{47.7}=41.9 \text{ feet per second}$$

$$h_v=\frac{41.9^2}{64.4}=27.3 \text{ feet}$$

From table *B–3* (appendix B), for 75 percent full flow, $\frac{d}{D}=0.702$ and $r=0.2964(9.0)=2.67$. Using a value of n of 0.018 to maximize the losses, by Manning's equation (equation (*30*), appendix B),

$$s=\left(\frac{vn}{1.486r^{2/3}}\right)^2=\left[\frac{(41.9)(0.018)}{(1.486)(2.67)^{2/3}}\right]^2=0.07,$$

and $h_f=0.07\times270=18.9$ feet.

The invert elevation at the downstream portal of the conduit will then be equal to (1) the elevation of the throat, plus (2) the velocity head at the throat, minus (3) the velocity head in the conduit flowing 75 percent full, minus (4) the friction losses in the conduit, minus (5) the depth of flow at the downstream portal. The required portal invert elevation for this trial conduit diameter is approximately:

$$\begin{aligned}\text{Invert elevation}&=93.1+\frac{1}{1.1}(110-93.1)-27.3-18.9-0.702(9.0)\\&=93.1+15.4-27.3-18.9-6.3=56.0\end{aligned}$$

Since the outlet portal invert elevation required for the 9.0-foot-diameter conduit is lower than the established elevation 60.0, this conduit is too small and a larger size must be selected.

Assume a new trial conduit diameter=9.50 feet at throat elevation 96.4.

The approximate tunnel length equals 273 feet. Assuming that the conduit flows 75 percent full,

$$\text{Area}=0.75(\pi)(4.75)^2=53.2 \text{ square feet}$$

$$\text{Velocity}=\frac{2{,}000}{53.2}=37.6 \text{ feet per second}$$

$$h_v=\frac{(37.6)^2}{64.4}=21.9 \text{ feet}$$

$$\frac{d}{D}=0.702 \text{ and } r=0.2964(9.5)=2.82.$$

Using Manning's equation,

$$h_f=273\left[\frac{(37.6)(0.018)}{1.486(2.82)^{2/3}}\right]^2=14.2 \text{ feet}$$

The required portal invert elevation for a 9.50-foot conduit diameter is approximately:

$$\begin{aligned}\text{Elevation}&=96.4+\frac{1}{1.1}(110-96.4)-21.9-14.2-0.702(9.5)\\&=96.4+12.4-21.9-14.2-6.7=66.0\end{aligned}$$

Although this elevation is somewhat higher than the established portal invert elevation 60, actual losses through the conduit will be larger than those estimated on the basis of the conduit flowing 75 percent full throughout its length. Therefore, the 9.50-foot-diameter conduit appears to be, for all practical purposes, the minimum uniform diameter conduit meeting the requirements of the problem. Computations of the water surface profile through the 9.50-foot-diameter conduit shown in table 25 are then performed in order to verify the approximate solution given above. These computations are based on Bernoulli's theorem (equation (32), app. B).

Discharge-head computations for this design are shown in table 26. For the lower range of heads the coefficient relationships of various $\frac{H_e}{R_s}$ values are obtained from figure 224, assuming a coefficient of 3.75 for an $\frac{H_e}{R_s}$ value of 0.3. For the higher ranges of head the discharges can be obtained from equation (29) using a throat radius of 4.75 at elevation 96.4. Smooth curves are then plotted for both head range computations. The intersection of the curves is replaced by an approximate transition curve to more nearly repre-

TABLE 25.—*Water surface profile computations for case 1 (9.5-foot-diameter conduit)*

[$Q=2{,}000$ second-feet. $n=0.018$]

Station	ΔL	Trial d/D	d	a	v	h_v	r	$r^{2/3}$	s	$\frac{s_1+s_2}{2}$	Δh_L	$\Sigma\Delta h_L$	$d_2+h_{v_2}+\Sigma\Delta h_L$	Invert elevation	Datum gradient	Remarks
1+00		1.00		70.9	28.2	12.4	2.37	1.78	0.037					96.4	108.8	
1+19	42	0.56	5.3	40.8	49.0	37.3	2.54	1.86	102	0.070	2.9	2.9	45.5	61.0	106.5	Too low
		.55	5.2	40.0	50.0	38.8	2.52	1.85	107	.072	3.0	3.0	47.0		108.0	OK
2+30	111	.60	5.7	44.4	45.1	31.6	2.64	1.91	.082	.095	10.5	13.5	50.8	60.5	111.3	Too high
		.61	5.8	45.3	44.2	30.3	2.66	1.92	.078	.092	10.2	13.2	49.3		109.8	OK
3+50	120	.70	6.6	53.0	37.7	22.1	2.81	1.99	.053	.066	7.9	21.1	49.8	60.0	109.8	Too high
		.71	6.7	53.8	37.2	21.5	2.83	2.00	.051	.065	7.8	21.0	49.2		109.2	OK.

TABLE 26.—*Computations for discharge curve for case 1*
$R_s=7.0$ feet

Head on crest, feet	Crest control			Throat control	
	$\frac{H_o}{R_s}$	C^*	$Q=C(2\pi R_s)H_o^{3/2}$	H_a	$Q=\left(\frac{R}{0.204}\right)^2 H_a^{1/2}$
1	0.14	3.56	157		
2	.29	3.75	368		
3	.43	3.58	820		
4				7.6	1,500
6				9.6	1,680
8				11.6	1,850
10				13.6	2,000

*Coefficient of 3.75 assumed for $\frac{H_o}{R_s}=0.3$ (from fig. 223). Coefficients for $\frac{H_o}{R_s}$ values other than 0.3 based on ratios shown on fig. 224.

sent actual conditions. The discharge curve is plotted on figure 232. The computations show that the conduit will be only 76 percent full at the downstream end; therefore the design is satisfactory.

Case 2.—Radius of crest to be determined to minimize subatmospheric pressures along crest.

First determine the minimum crest radius for the given $H_o=10$ feet and $Q=2{,}000$ second-feet similar to case 1. Assume $\frac{P}{R_s}=0.15$ and, as in case 1, by trial and error determine value of R_s.

Assume $R_s=7.0$ feet, $\frac{H_o}{R_s}=\frac{10}{7}=1.43$. For $\frac{H_o}{R_s}=1.43$ and $\frac{P}{R}=0.15$ from figure 223, $C_o=1.55$. Then $Q=C_o(2\pi R_s)H_o^{3/2}=1.55\ (2\pi)\ (7.0)\ (10)^{3/2}=2{,}155$ second-feet. Since a 2,000-second-foot discharge is required, the assumed value of R_s is too large.

Assume $R_s=6.7$ feet. $\frac{H_o}{R_s}=\frac{10}{6.7}=1.49$. From figure 223 $C_o=1.49$ and $Q=1{,}985$ second-feet, which is approximately the required discharge.

484556 O—60——23

Using the value of $\frac{H_o}{R_s}$ thus computed, enter figure 228 and find the approximate increased crest radius required to minimize subatmospheric pressures. For $\frac{H_o}{R_s}=1.49$, $\frac{R_s'}{R_s}=1.74$, and $R_s'=1.74(6.7)=11.7$ feet; use 11.75 feet. Points on the profile of the crest shape which conform to the lower nappe surface for an $\frac{H_s'}{R_s'}=0.30$ and $R_s'=11.75$ are computed using values from table 24 and are plotted as shown on figure 231.

Computations for the required transition shape to pass 2,000 second-feet with a head of 10 feet on the crest are identical to those given in case 1. Figure 231 shows the plotted points and the crest and transition curves.

From an inspection of the plotting of the transition and crest shapes for case 2, it can be seen that the conduit diameter for case 1 is too large for case 2. If the 9.5-foot-diameter conduit used in case 1 were used in case 2, a smooth transition connecting the crest and conduit would be considerably outside the transition shape determined by equation (29). This means that for a head of 10 feet on the crest, the discharge would no longer be limited to 2,000 second-feet by the transition but would increase because of the larger size transition. The resulting discharge would be approximately 2,400 second-feet. This discharge not only exceeds the maximum discharge assumed in the problem, but also would require a larger uniform-diameter conduit in order to pass the discharge and not flow more than 75 percent full. A still larger uniform-diameter conduit with a still larger maximum discharge would finally have to be adopted to obtain a satisfactory hydraulic design. If, however, a smaller uniform-diameter

conduit is chosen, it is apparent that the conduit will flow more than 75 percent full at the downstream end.

The simplest solution to this problem is to vary the diameter of the conduit. An upstream diameter is chosen based on the crest profile and transition where they converge. This procedure establishes a throat of the size necessary to limit the maximum discharge to 2,000 second-feet. At some suitable location downstream from the throat, the conduit is enlarged to prevent it from flowing more than 75 percent full. Location of the enlargement is determined by economic or construction considerations to meet hydraulic requirements.

For this problem, a conduit diameter of 8.25 feet is selected with the throat at elevation 86.0. It will be assumed that the most economical design is obtained by extending the 8.25-foot-diameter conduit to the point where it flows 75 percent full, at which point the conduit is enlarged to the diameter needed to make it flow 75 percent full at the downstream portal. In order to determine the point at which the tunnel must be enlarged, water surface profiles are run downstream by the step method as shown in table 27. A bend radius of 16.5 feet ($2D$) is used. The table shows that the conduit must increase in size starting at the P.T. of the vertical bend, station 1+16.5. The downstream size of the conduit may be approximated by assuming a given size conduit flowing 75 percent full at the downstream portal and using the distance from enlargement to portal as one reach in the water surface profile computations. Although this method results in losses slightly larger than would be obtained by using shorter reaches, it is sufficiently accurate to determine conduit size if the length of the conduit downstream from the expansion is not excessively long. Use of shorter steps and an assumed minimum value of n would be required to determine the depth and velocity at the downstream portal for use in designing an energy dissipator. The transition from the smaller to the larger diameter conduit should be proportioned as explained in section 197(b).

TABLE 27.—*Water surface profile computations for case 2 (varying diameter conduit)*

[Q=2,000 second-feet. n=0.018]

Station	ΔL	Trial d/D	d	a	v	h_v	r	$r^{2/3}$	s	$\frac{s_1+s_2}{2}$	Δh_L	$\Sigma\Delta h_L$	$d_2+h_{v_2}+\Sigma\Delta h_L$	Invert elevation	Datum gradient	Remarks
1+00		1.00		53.5	37.4	21.7	2.06	1.62	0.078					86.0	107.7	
1+16.5	30	0.6	5.0	33.5	59.7	55.4	2.29	1.74	.173	0.125	3.8	3.8	64.2	61.0	125.2	
		.7	5.8	40.0	50.0	38.9	2.44	1.81	.112	.095	2.9	2.9	47.6		108.6	OK.
Try 9.50-foot-diameter conduit flowing 75 percent full at the portal																
3+50	234	.702	6.7	53.1	37.7	22.1	2.81	1.99	.052	.082	19.2	22.1	50.9	60.0	110.9	Too high.
Try 9.75-foot-diameter conduit flowing 75 percent full at the portal																
3+50	234	.702	6.8	56.0	35.7	19.8	2.89	2.03	.045	.078	18.3	21.2	47.8	60.0	107.8	OK.

Discharge-head relationships for this case are computed similarly to those for case 1. The throat radius in this instance is 4.13 feet at elevation 86.0. Computations are shown in table 28. This discharge curve is plotted on figure 232.

206. *Culvert Spillways.*—(a) *General.*—As described in section 186(h), the culvert spillway ordinarily consists of a simple culvert conduit placed through the dam or along the abutment, generally on a uniform grade with the entrance placed vertically or inclined. The culvert cross section might be round if it is constructed of fabricated or precast pipe, or it might be square, rectangular, or of some other shape if cast in place. The culvert can freely discharge, or it can empty into an open channel so that the outflowing jet is supported along the channel floor.

The factors which combine to determine the nature of flow in a culvert spillway include such variables as the slope, size, shape, length, and roughness of the conduit barrel, and the inlet and outlet geometry. The combined effect of

Table 28. *Computations for discharge curve for case 2* $R_s' = 11.75$ *feet*

Head on crest, feet	Crest control			Throat control	
	$\frac{H_s}{R_s'}$	C *	$Q = C(2\pi R_s')H_s^{3/2}$	H_a	$Q = \left(\frac{R}{0.204}\right)H_a^{1/2}$
1	0.09	3.55	260		
2	.17	3.74	780		
3	.26	3.85	1,480	17	1,680
4	.34	3.82	2,260	18	1,730
6				20	1,830
8				22	1,920
10				24	2,000

*Coefficient of 3.80 assumed for $\frac{H_s}{R_s} = 0.3$ (from fig. 223). Coefficients for $\frac{H_s}{R_s}$ values other than 0.3 based on ratios shown on fig. 224.

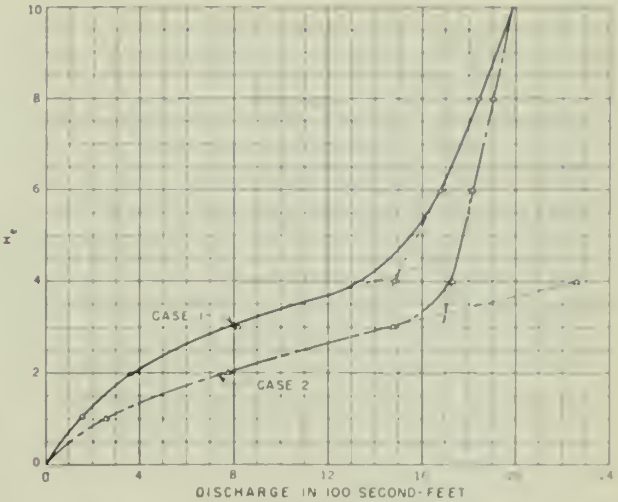


Figure 232. **Drop inlet spillway—discharge curves for illustrative example.**

these factors determines the location of the control which in turn determines the discharge characteristics of the conduit. The location of the control dictates whether the conduit flows partly full or full and thereby establishes the head-discharge relationship.

The grade of the conduit might be mild or steep; that is, its slope may be flatter or steeper than one which for a given discharge will just support flow at the critical stage. For either the mild or steep slope the control may be either at the inlet or the outlet, depending on the entrance geometry and head relationship and on the flow conditions at the outlet. The various conditions which may govern a particular flow are illustrated on figure 233.

If the inlet is not submerged the control for a conduit on a mild slope flowing partly full will usually be at the outlet. If the outlet discharges freely the flow at that point will pass through critical depth. This condition is shown as ① in figure 233. If the tailwater is high enough to maintain a depth greater than critical, the tailwater level will control the flow in the upstream barrel. If the tailwater submerges the outlet, the conduit might flow full for its entire length and thus submerge the inlet. This flow condition is depicted as ⑥ on figure 233. Until the conduit flows full, the flow ordinarily will be at subcritical stage and the discharge relationships will be determined according to the Bernoulli equation with computations starting at the outlet

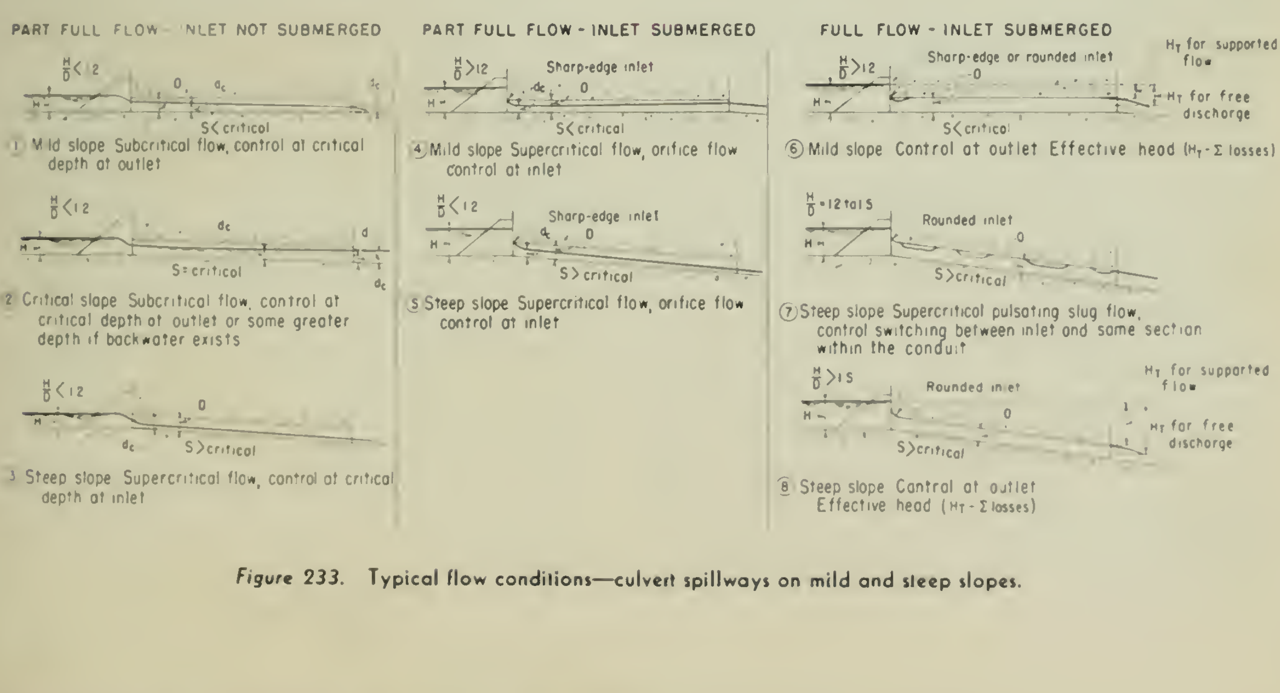


Figure 233. **Typical flow conditions—culvert spillways on mild and steep slopes.**

Where the reservoir level submerges the inlet and where $\frac{H}{D}$ exceeds 1.2, the control at critical depth may occur at the inlet, if the culvert is relatively short so that a jump does not form within the barrel. This condition is illustrated as ④.

When the conduit is on a steep slope and the entrance is not submerged, the flow will be controlled by critical depth at the inlet, as indicated by condition ③ on figure 233. The water surface will drop rapidly to critical depth at the entrance, and open channel flow at supercritical velocities will exist throughout the conduit barrel. Discharge for a given reservoir level will be governed by channel or weir flow, assuming critical depth to occur at the culvert entrance.

After the inlet has been submerged or where H exceeds about $1.2D$, it is still possible to have open channel flow at supercritical stage in the conduit barrel, as depicted for condition ⑤, if the control remains at the entrance. In this case flow at the inlet is analogous to orifice or sluice flow. This flow condition is contingent on the formation of a contraction at the top of the entrance so that an airspace is maintained along the top of the barrel to permit part full flow throughout the conduit length.

As the head at the entrance and the resulting discharge increase, channel friction or local disturbances may force the barrel to flow full near the outlet, thus sealing the conduit at the downstream end. The high-velocity flow in the culvert will carry away some of the air trapped at the top of the barrel, thus reducing the pressure in the conduit to less than atmospheric. Furthermore, if the entrance is shaped to eliminate the inlet contraction, the barrel will start to flow full near the inlet, after which the full flow zone will extend rapidly down the conduit toward the outlet. The effect of the full flow condition will be a draft tube action (similar to siphonic action) which will increase the discharge. The increased discharge will cause a deeper drawdown just upstream from the inlet. A vortex will form and air which will thus be introduced into the culvert will break the draft tube action. The reduction in discharge will result in the return to orifice control at the inlet. Immediately, the full flow action will begin again and the cycle will be repeated. This alternate priming and breaking action will cause a pulsating flow stage with the slug flow phenomenon indicated by condition ⑦ on figure 233. When the reservoir stage condition is such that the $\frac{H}{D}$ ratio exceeds 1.5, the entrance drawdown may be insufficient to interfere with the full flow action, and a steady state of full pipe flow indicated by condition ⑧ will prevail.

If it is intended that the spillway conduit not flow full, the geometry of the inlet becomes an important consideration. The inlet must be shaped to obtain a maximum discharge efficiency and yet maintain a top contraction which will provide a freely aerated surface in the conduit barrel for all reservoir stages. The sharp-edged square inlet produces the desired contraction without materially reducing the discharge capacity. The inlet contraction can also be formed (but at reduced hydraulic capacity) by a projecting inlet, a mitered inlet with a downstream sloping face, an inlet orifice ring which is made smaller than the remainder of the conduit, or a curtain wall closing off the top of the conduit entrance.

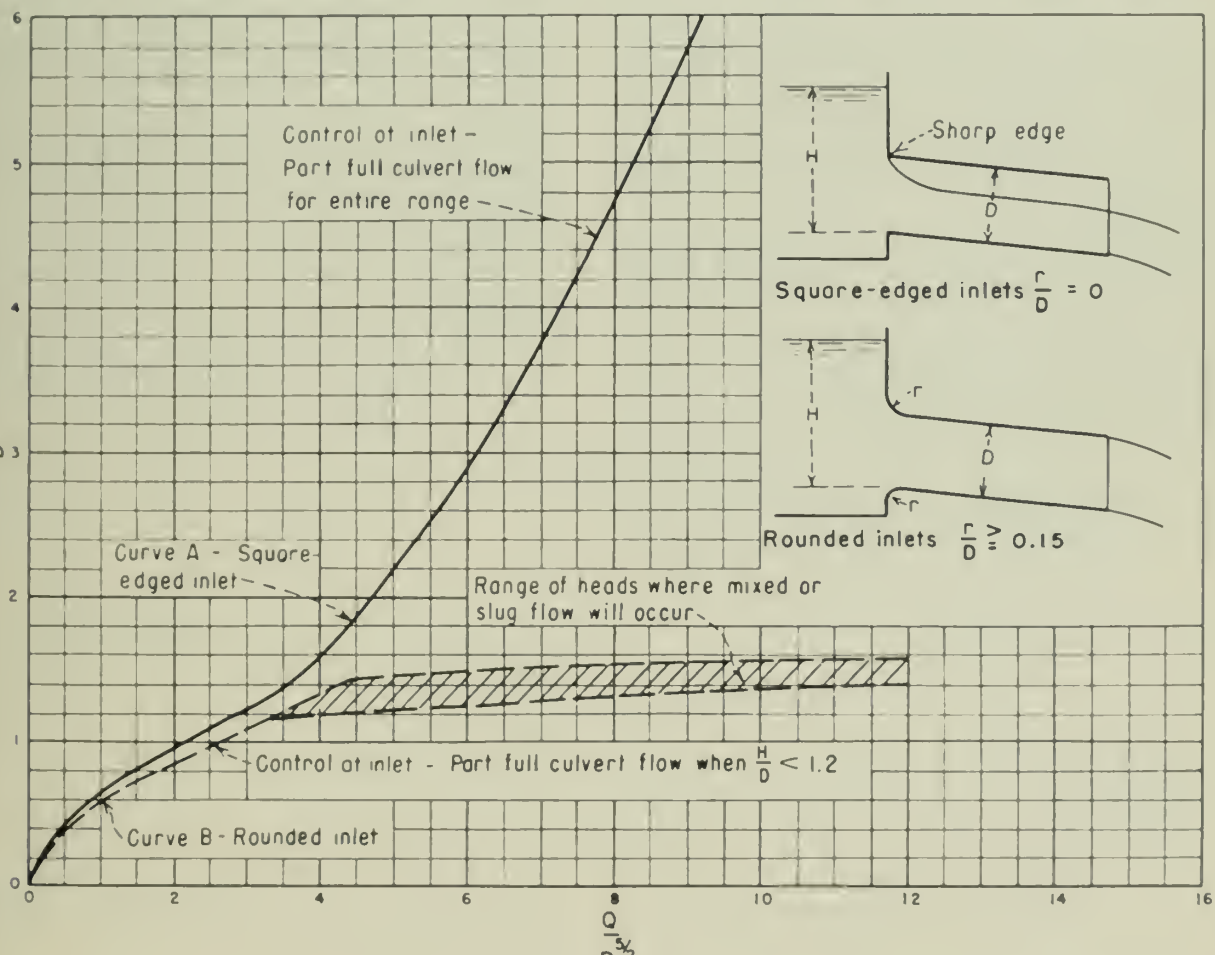
If the conduit is permitted to flow full at the higher reservoir stages, the control will be at the outlet and the geometry of the inlet has much less significance. For this case the inlet must be shaped to minimize the jet contraction to avoid separation of the incoming flow from the conduit barrel, since it is desired to obtain full pipe flow for all conditions except when the inlet is not submerged. The more streamlined shape will reduce entrance losses for the full pipe flow condition. The suppression of the contraction is achieved by rounding the inlet or by providing a gradually tapering transition to the conduit barrel.

Culvert inlets may have many variations in approach conditions, in cross-sectional shapes, and in entrance arrangements. For example, an entrance may be rounded, beveled, square or bellmouthed; it may be installed either flush with or protruding through a vertical or sloping headwall. The approach to the inlet may or may not be a well-defined channel. Wing walls or warped transition approaches may be utilized. In cross section, a culvert entrance may be round, square, rectangular, or arch shaped. All such variations have a marked bearing on the culvert performance, as they affect orifice discharge, inlet contractions, and the entrance losses for full pipe flow.

A common arrangement for a circular pipe culvert installation provides for a vertical headwall with the pipe end placed flush with the wall. Similarly, box culvert arrangements usually involve a trapezoidal approach channel with vertical or warped approach walls leading to the culvert entrance. The hydraulic design of these two types of installation is discussed in detail.

(b) *Circular Conduit with Vertical Headwall.*—Figure 234 shows head-discharge relationships for a circular conduit placed flush with a vertical headwall, for both square-edged and rounded inlets. This plotting is based on an average of numerous experimental tests [16, 17, 18, 19] of pipe culvert entrances with the conduit placed on steep slopes. The head-discharge relationships for the square-edged inlet are based on the control remaining at the inlet for all reservoir heads. Where the $\frac{H}{D}$ values are less than about 1.2, the flow characteristics are those of critical depth flow in a circular pipe, modified only by the effects of the jet contraction. For $\frac{H}{D}$ ranges above 1.2 the flow characteristics are those of orifice or sluice flow. Since the conduit is considered to be flowing partly full at supercritical stage for all $\frac{H}{D}$ ranges indicated, the downstream conditions will have no influence on the discharge.

On figure 234 the head-discharge relationships for the rounded inlet for values of $\frac{H}{D}$ less than about 1.2 lie to the right of those for the square-edged inlet, indicating slightly greater discharges for equal size conduits. The increased discharge



Adapted from reference [17]

Figure 234. **Head-discharge curves for square-edged and rounded inlets for circular culverts on steep slopes.**

capacity through the critical-depth flow range is the result of improved streamlined flow brought about by the suppression of the inlet contractions. For $\frac{H}{D}$ values exceeding 1.2 the pulsating flow characteristics will begin and the discharge-head relationship in this range of flow will be uncertain; it cannot be determined until the flow stabilizes as full flow stage. Since full pipe flow is governed by control at the outlet, the head-discharge relationship can be determined by the application of Bernoulli's theorem. Referring to figure 235,

$$H_T=H+L\sin\theta-\frac{D}{2} \tag{30}$$

Similarly,

$$H_T=h_v+h_e+h_f, \text{ or}$$

$$H_T=\left(1+K_e+f\frac{L}{D}\right)\frac{v^2}{2g} \tag{31}$$

where K_e is the entrance loss coefficient and f is the friction loss coefficient in the Darcy-Weisbach formula, $h_f=f\frac{L}{D}\frac{v^2}{2g}$ (see secs. 227 and 228).

Combining equations (30) and (31), dividing by D, and stating the equation in terms of Q instead of v,

$$\frac{H}{D}+\frac{L}{D}\sin\theta-\frac{1}{2}=0.0252\left(1+K_e+f\frac{L}{D}\right)\left(\frac{Q}{D^{5/2}}\right)^2 \tag{32}$$

In the above equation, it is assumed that the culvert discharges freely at the outlet and that the pressure line at the outlet is approximately at the center of the pipe. If the outlet discharges into a channel so that the outflowing jet is supported, equation (32) becomes:

$$\frac{H}{D}+\frac{L}{D}\sin\theta-1.0=0.0252\left(1+K_e+f\frac{L}{D}\right)\left(\frac{Q}{D^{5/2}}\right)^2 \tag{33}$$

Equations (32) and (33) are for full flow conditions. They are expressed in terms of $\frac{H}{D}$ and $\frac{Q}{D^{5/2}}$ so that by referring to figures 233 and 234 it can be determined whether or not the full flow condition exists.

Where friction losses are to be based on values of Manning's friction factor, n, the value of f in the Darcy-Weisbach formula can be determined from figure B–7 (app. B). Appropriate values of n are given in section 228(b). Values of K_e for various entrance conditions have been determined by different experimenters, with results ranging in the values shown below:

	Range	*Average*
For square-edged inlets installed flush with vertical headwalls.	0.43 to 0.70	0.50
For rounded inlets installed flush with vertical headwalls, $\frac{r}{D}\geq 0.15$.	0.08 to 0.27	.10
For grooved or socket-ended concrete pipe installed flush with vertical headwall.	0.10 to 0.33	.15
For projecting concrete pipe with grooved or socket ends.		.20
For projecting steel or corrugated metal pipes.	0.5 to 0.9	.85

Nomographs for determining flow in circular pipes with entrance control and flowing full,

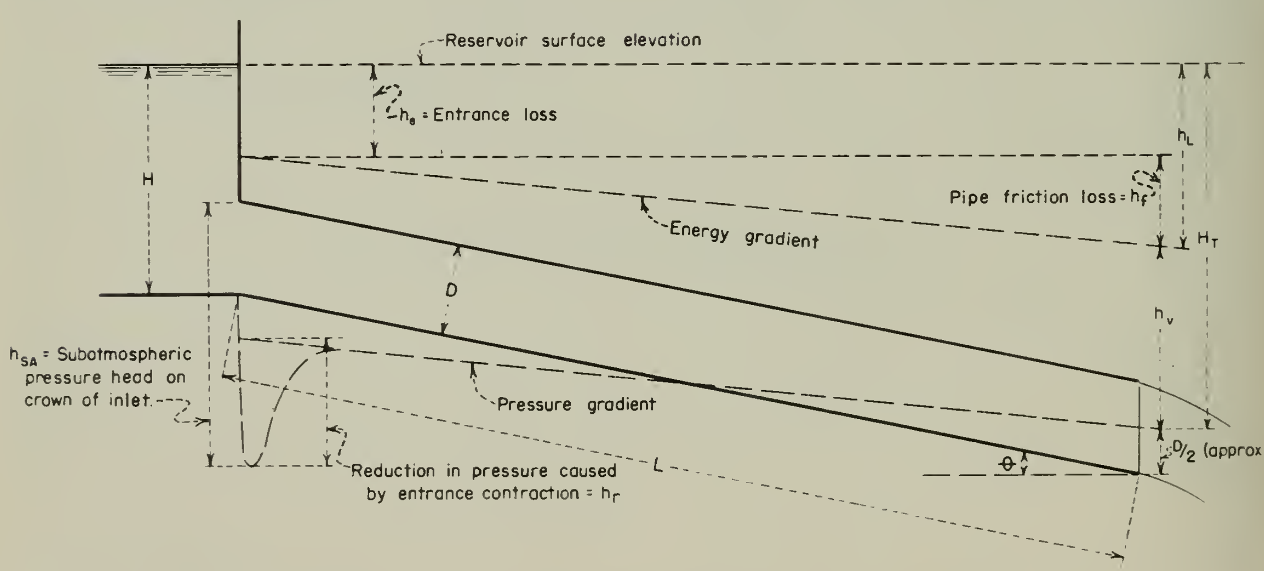


Figure 235. **Hydraulic characteristics of full pipe flow for culvert spillways.**

which have been developed by the U.S. Bureau of Public Roads, are included in appendix B. These can be used as design aids in determining flow in circular culvert spillways. Figure B–8 is for flow in concrete pipe culverts having entrance control and the following types of entrances: (1) Headwall with square-edge entrance, (2) headwall with groove-end pipe, and (3) headwall with groove end of pipe projecting. Figure B–9 is for flow in corrugated metal pipe culverts having entrance control and the following types of entrances: (1) Flush headwall, (2) end mitered to conform to slope, and (3) projecting pipe. Figure B–10 is for concrete pipe culverts flowing full, based on a value of $n=0.012$ and entrance loss coefficients of 0.1, 0.2, and 0.5. Figure B–11 is for corrugated metal pipe flowing full, based on a value of $n=0.024$ and entrance loss coefficients of 0.5 and 0.9.

(c) *Box Culvert with Vertical or Warping Inlet Walls.* If the inlet is such that the bottom and side contractions will be suppressed, flow through a box culvert on a steep slope can alternately go through the three distinct phases of flow described previously, depending on submergence conditions and on the factors which dictate flow conditions within the conduit.

For conditions when the inlet is not submerged, critical flow will occur in the region of the inlet, in which case for a rectangular section,

$$d_c=\sqrt[3]{\frac{q^2}{g}} \text{ or } H=1.5\sqrt[3]{\frac{q^2}{g}}$$

Relating this equation of critical flow to the discharge Q,

$$Q=w\sqrt{g}\left(\frac{H}{1.5}\right)^{3/2} \qquad (34)$$

where w is the width at the culvert entrance.

When the conduit entrance becomes submerged, the flow may be considered analogous to that of a sluice if the entrance has a square edge at the top. For this condition, top contraction of the jet will occur, and flow can be computed according to orifice flow, or $Q=CA\sqrt{2gh}$. The coefficient C depends on whether the area A is defined as the area of the opening, the area of the contracted jet, or some similar referenced area. Similarly, the coefficient will depend on the definition of the head h, whether it is measured to the top, center, or bottom of the opening. Ordinarily, for a square-edged orifice in a vertical headwall the

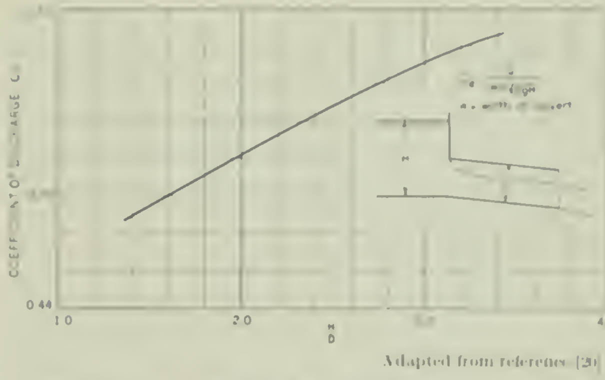


Figure 236. Coefficient of discharge for submerged box culvert spillway with square-edged top opening.

area is measured at the plane of the headwall face (designated as a). If the head is measured from the water surface to the bottom of the opening (designated as H), the discharge can be computed by the equation:

$$Q=C_d a\sqrt{2gH}, \text{ or } Q=C_d wD\sqrt{2gH} \qquad (35)$$

where D is the height of the opening. Values of C_d as determined from experiments [20] are plotted on figure 236.

As with circular culverts, full flow in box culverts depends on suppression of the top contraction. Full culvert flow will be governed by control at the outlet, and discharge-head relationships can be computed according to the equation:

$$Q=a\sqrt{2g(H_T-h_L)} \qquad (36)$$

where a is the area of the culvert barrel and H_T and h_L are heads as indicated on figure 235.

Reducing the equation and expressing it in terms of the entrance loss coefficient, K_e, and of the friction loss coefficient, Manning's n,

$$Q=a\sqrt{2g}\left[\frac{H+L\sin\theta-\frac{D}{2}}{1+K_e+\frac{29.1n^2L}{r^{4/3}}}\right]^{1/2} \qquad (37)$$

where r is the hydraulic radius of the culvert flowing full. The above equation is based on free discharge at the outlet. If the outflowing jet is supported, equation (37) will become:

$$Q=a\sqrt{2g}\left[\frac{H+L\sin\theta-D}{1+K_e+\frac{29.1n^2L}{r^{4/3}}}\right]^{1/2} \qquad (38)$$

Bureau of Public Roads nomographs for solution of flow in box culverts are also included in

appendix B. Figure B–12 is for box culverts with entrance control for various positions of the wingwalls. The discharges are based on discharge coefficients which approximate those shown on figure 236. Figure B–13 is for concrete box culverts flowing full, based on a value of $n=0.013$, and entrance loss coefficients of 0.1, 0.2, 0.5, and 0.7

(d) *Conduit Pressures.*—When the grade of a culvert spillway is greater than the friction slope, for full pipe flow the pressure gradient will lie below the center of the pipe, as indicated on figure 235. The difference in head between this hydraulic gradient and any point on the pipe vertically above it will be the measure of the subatmospheric pressure that will exist at the point. Cavitation will occur when the subatmospheric pressure approaches one atmosphere, so that the residual absolute pressure is near vapor pressure. Collapsing can occur whenever the residual pressure in the pipe is less than atmospheric. To avoid cavitation along the pipe surfaces, the minimum absolute pressure must be limited to some value greater than vapor pressure. The pressure reduction in the pipe will be greatest at the crown immediately downstream from the entrance. It can be reduced further by any pressure drop caused by an inlet contraction, such as a sharp-edged or constricted opening.

From figure 235 it can be seen that

$$h_v+h_e+h_r=h_{SA}+(H-D) \qquad (39)$$

where:

h_r = the reduction of pressure head due to contraction, and

h_{SA} = the resulting subatmospheric pressure head.

Vapor pressure of water varies with temperature. It is equivalent to about 0.2 foot of head at 32° F. and about 1.4 feet of head at 85° F. To make sure that cavitation is avoided and to allow for other uncertainties, the residual pressure ordinarily should not be significantly less than 10 feet absolute. Based on probable minimum atmospheric pressures at different elevations above sea level, the limiting subatmospheric pressures indicated in table 29 are recommended.

The reduction in pressure head due to jet contraction will depend on the geometry of the inlet. For streamlined entrances very little reduction will be effected, but for sharp-edged

TABLE 29.—*Allowable subatmospheric pressures for conduits flowing full*

Elevation above sea level	Allowable subatmospheric pressure, h_{SA}, in feet of water
0	22
2000	20
4000	18
6000	16
8000	14

projecting inlets the reduction can be almost equal to the velocity head. For sharp-edged square inlets the reduction in pressure may approach $0.7h_v$. Equation (39) written in terms of loss coefficients (sec. 227) will be:

$$\frac{v^2}{2g}(K_v+K_e+K_r)=h_{SA}+(H-D), \text{ or} \qquad (40)$$

$$\frac{v^2}{2g}=h_v=\frac{h_{SA}+(H-D)}{K_v+K_e+K_r} \qquad (41)$$

where K_r is the pressure reduction coefficient.

For a square-edged entrance where $K_e=0.5$, $K_r=0.7$, $K_v=1.0$, $H=1.5D$, and h_{SA} (for an installation at 6,000 feet above sea level) is 16 feet, equation (41) will be written:

$$h_v=\frac{16+0.5D}{1.0+0.5+0.7}=\frac{16+0.5D}{2.2}$$

For a 4-foot-diameter conduit, the velocity head is 8.2 feet and the velocity in the conduit would have to be limited to about 23 feet per second. From equation (31) the total drop from the reservoir water surface to the centerline of the downstream end of a 200-foot-long conduit with an f value of 0.023 for $D=4$ (figure B–7, appendix B, for a value of $n=0.014$) is:

$$H_T=8.2\left(1.5+f\frac{L}{D}\right), \text{ or}$$

$$H_T=8.2\left(1.5+\frac{0.023\times200}{4.0}\right)=21.7 \text{ feet.}$$

(e) *Antivortex Devices.*—Although experimental tests have shown that for a properly rounded entrance the culvert begins to flow full after the $\frac{H}{D}$ ratio exceeds 1.2, the full pipe flow condition could not be stabilized until an $\frac{H}{D}$ value of 1.5 or

higher was reached. This condition was due to the "slug flow action" which resulted from the introduction of air into the conduit by entrance drawdown and by vortices immediately upstream from the inlet. To reduce the range where slug flow action prevails, antivortex devices have been used above conduit entrances. These devices not only stabilize the flow condition at a lower $\frac{H}{D}$ ratio, but they also help to start the priming action sooner. The devices have consisted of grillages, rafts, or fixed solid hoods placed so as to break up the vortices or to prevent their formation where they could feed air into the conduit [24]. In order to be effective, the hood or grillage must be placed immediately above the entrance and must extend at least two diameters in front of and to each side of the inlet.

(f) *Energy Dissipators.*—A culvert spillway may discharge freely or it may empty into an open channel chute which then conveys the flow to a downstream terminal structure. The flow from a freely discharging conduit might empty directly into the natural stream channel, into a trapezoidal plunge basin described in section 230(b), or into an impact basin described in section 202. Where the discharge from the full flowing culvert empties into an open chute, the hydraulics beyond the portal will be according to open channel flow, as discussed in section 197. Stilling devices such as those described in sections 198 through 203 can be utilized to dissipate the energy of flow before returning the discharge to the river channel.

(g) *Design Examples.*—To illustrate the procedures for a culvert spillway design, several typical examples are presented.

Example 1.—The size of a culvert spillway required to discharge 100 second-feet at reservoir elevation 110 is to be determined. The normal sill level of the spillway entrance is at elevation 100. The culvert is to flow partly full for all heads. If a circular conduit is selected, the design procedure is as follows:

The head-discharge-diameter relationship for a circular conduit with entrance placed flush with a vertical headwall can be obtained from figure 234. Curve A is used because the conduit is to flow partly full. By assuming various sizes of conduit, a size can be found which meets the requirements, as follows:

Assume a conduit 3.5 feet in diameter, then $D^{5/2}=23$ and $\frac{Q}{D^{5/2}}=4.35$. For a $\frac{Q}{D^{5/2}}$ value of 4.35, $\frac{H}{D}=1.75$ and $H=6.1$. Since an H of 10 feet is available, the culvert can be made smaller.

As a second trial, assume a 3-foot conduit. Then $D^{5/2}=15.6$ and $\frac{Q}{D^{5/2}}=6.41$. From curve A, $\frac{H}{D}=3.2$ and $H=9.6$, which approximates the 10 feet available.

If a box culvert is selected, the procedure is as follows:

$$Q=C_d wD\sqrt{2gH} \text{ (equation (35))}$$

$$wD=\frac{Q}{C_d\sqrt{2gH}}$$

Assuming a 2.5-foot-high culvert, $\frac{H}{D}=\frac{10}{2.5}=4.0$, and C_d from figure 236 is approximately 0.6. Then

$$2.5\,w=\frac{100}{0.6\times 8.02\times\sqrt{10}}=6.6, \text{ and } w=2.6 \text{ feet.}$$

Example 2.—Find the discharge through the conduits in the previous example if the entrances are shaped to provide full conduit flow. The conduit length is 200 feet and the invert grade at the outlet is elevation 80.0. The conduit discharges freely at the outlet end. The procedure is as follows:

Equation (32) may be written:

$$\left(\frac{Q}{D^{5/2}}\right)^2=\frac{1}{0.0252}\left[\frac{\frac{H}{D}+\frac{L}{D}\sin\theta-\frac{1}{2}}{1+K_e+f\frac{L}{D}}\right]$$

For a 3-foot circular conduit with K_e of 0.10 and $f=0.025$ (from figure B–7, appendix B, for a value of $n=0.014$),

$$\left(\frac{Q}{15.6}\right)^2=\frac{1}{0.0252}\left[\frac{\frac{10}{3}+\frac{200}{3}\times\frac{20}{200}-\frac{1}{2}}{1+0.10+\frac{0.025\times 200}{3.0}}\right]=136$$

and $\frac{Q}{15.6}=11.67$. Then $Q=182$ second-feet. This flow will provide a velocity of 25.7 feet per second in the conduit.

Equation (40) may be written as:

$$h_{SA}=\frac{v^2}{2g}(K_v+K_e+K_r)-(H-D).$$

The subatmospheric pressure in the conduit, based on a pressure reduction coefficient, K_r, of 0.1 and a value of K_e of 0.1 for a rounded entrance is equal to:

$$\frac{(25.7)^2}{64.4}(1.0+0.1+0.1)-(10.0-3.0)=5.3 \text{ feet.}$$

This subatmospheric pressure is less than that allowed in table 29; therefore the design is satisfactory.

For the box culvert spillway, from equation (37), assuming $K_e=0.1$ and $n=0.014$,

$$Q=2.5\times2.6\times8.02\left[\frac{10+200\times\frac{20}{200}-1.25}{1+0.1+\frac{29.1\times200(0.014)^2}{(0.64)^{4/3}}}\right]^{1/2}$$

$$=157 \text{ second-feet.}$$

The velocity in the culvert will then be about 24 feet per second.

207. *Siphon Spillways.*—After it primes, flow in a siphon spillway is comparable to that in a culvert spillway and the hydraulic design criteria for the latter will apply with only slight variations. As with the culvert spillway, the design of the siphon spillway requires (1) the establishment of the minimum allowable absolute pressures, (2) the determination of the maximum corresponding throat velocity, and (3) the evaluation of various losses throughout the closed pipe system.

To determine the pressure conditions at the crest of the throat of the siphon, equation (39) can be rewritten as follows:

$$h_{v_s}=h_{SA}+h_s-\Sigma h_{L_u} \qquad (42)$$

where:

h_{v_s} = the velocity head at the crest of the throat,

h_{SA} = the subatmospheric pressure head at the crest of the throat (the maximum allowable subatmospheric pressure head is equal to the probable minimum atmospheric pressure head at the location under consideration, minus the minimum tolerable residual pressure head),

h_s = the head measured from the reservoir water level to the crest of the throat, for the condition when the siphon primes, and

Σh_{L_u} = the summation of all head losses upstream from the throat.

Subatmospheric pressure heads in excess of those indicated in table 29 should be avoided at the throat because lower pressures increase the tendency of the flow to part from the crest, forming vapor pressure pockets which induce cavitation along the siphon barrel walls. Such action is accompanied by undesirable vibration.

Flow around the upper bend of a siphon is similar to the action of free vortex flow in which the water stream rotates concentrically around a central axis. For vortex flow, the velocities of the flow elements vary inversely with increased distances from the center of rotation. Expressed mathematically,

$$vR=\text{A constant} \qquad (43)$$

Thus it can be seen that the velocity in the upper bend of the siphon will be greatest at the inside of the bend which is at the crest of the throat.

Equation (43) can be written:

$$v_xR_x=v_sR_c \text{ or } v_x=\frac{v_sR_c}{R_x} \qquad (44)$$

where v_x is the velocity at R_x distance from the center of curvature, and v_s is the velocity at the crest of the throat where the radius is R_c. The discharge, dq, of an element of flow of depth dR_x in the bend is equal to v_xdR_x. Substituting the value of v_x from equation (44),

$$dq=\frac{v_sR_c}{R_x}dR_x.$$

Integrating between the limits R_c and R_s, where R_s is the radius of curvature at the summit of the throat, results in the following equation for unit discharge:

$$q=v_sR_c\int_{R_c}^{R_s}\frac{1}{R_x}dR_x=v_sR_c\log_e\frac{R_s}{R_c} \qquad (45)$$

Substituting $\sqrt{2gh_{r_s}}$ for v_s, equation (45) can be written:

$$Q=8.02\,B\sqrt{h_{r_s}}R_c\log_e\frac{R_s}{R_c} \qquad (46)$$

where B is the width of the throat section.

Thus it can be seen that the maximum allowable discharge for a given value of h_{r_s} will depend on the curvature of the upper bend. Once the curvature is established, the remaining siphon components must be proportioned so that the limiting residual pressure head at the crest is maintained. Table 29 gives allowable subatmospheric pressures. For simplification, the allowable subatmospheric pressure can be taken as 0.7 of the probable minimum atmospheric pressure, h_{AT}. Equation (46) then may be written as:

$$Q=8.02\,B\sqrt{0.7h_{AT}}R_c\log_e\frac{R_s}{R_c} \qquad (47)$$

The discharge through a siphon depends on the available head from reservoir level to outlet level, less the accumulated head losses, including entrance loss, friction loss, losses due to transitions and bends, and the head lost at the exit. The above can be stated and solved with expressions similar to equations (5) and (7) in chapter IX, and by relating the velocity heads for the various component losses to that at the throat. The throat area is used for area a_1 in the equations.

Equation (36) expresses the discharge-head relationship of a closed conduit flowing full, where the area of the passageway is constant. For a rectangular siphon passageway where the areas of the upper and lower legs are the same as that of the throat, equation (36) can be written:

$$q=D\sqrt{2g(H_T-h_L)} \qquad (48)$$

where D is the height of the throat and siphon barrel.

Loss coefficients ordinarily assumed for a typical rectangular siphon similar to that shown on figure 185, where the main siphon waterway is of constant section, are as follows:

Entrance and converging transition loss,

$$h_e=0.2\frac{v_1^2}{2g}$$

Friction loss, $h_f=0.25\dfrac{v_1^2}{2g}$

Bend losses for upper and lower bends where the centerline radius $=2.5D$, $h_b=0.42\dfrac{v_1^2}{2g}$

Transition losses—diverging outlets,

$$h_{ex}=0.2\left(\frac{v_1^2}{2g}-\frac{v_o^2}{2g}\right)$$

Transition losses—converging outlets,

$$h_{ex}=0.1\left(\frac{v_o^2}{2g}-\frac{v_1^2}{2g}\right)$$

Exit losses, $h_v=\left(\dfrac{a_1}{a_o}\right)^2\dfrac{v_1^2}{2g}$

where a_1 and v_1 are the area and velocity, respectively, at the throat, and a_o and v_o are the area and velocity, respectively, at the outlet.

Equation (48), $q=D\sqrt{2g(H_T-h_L)}$, can be written in the form:

$$q=CD\sqrt{2gH_T} \qquad (49)$$

where C is a coefficient which will account for the various losses through the siphon passageway. Equating these two expressions for q and solving for C,

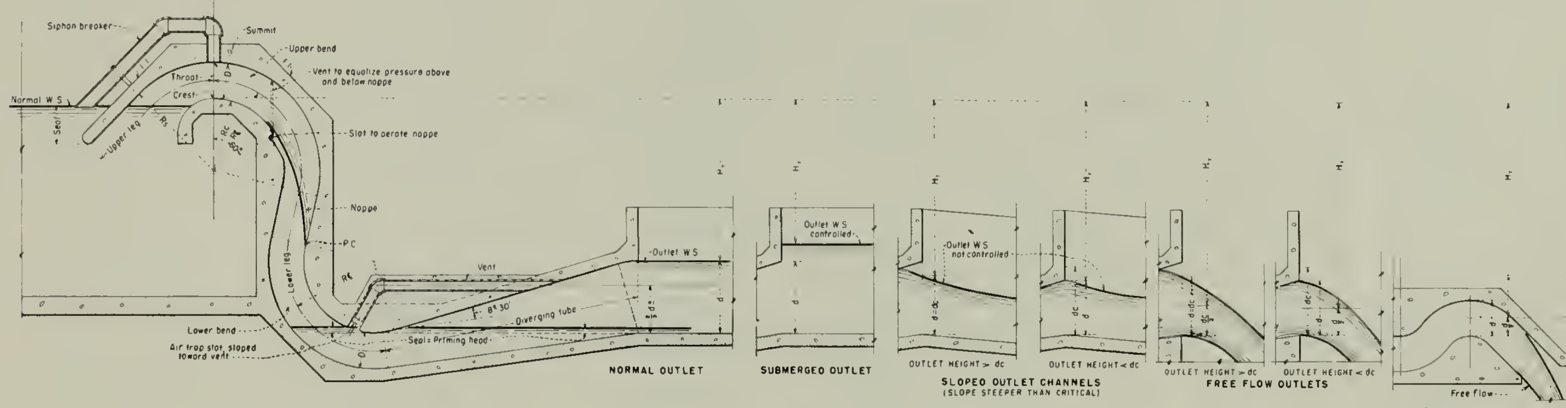
$$C=\sqrt{\frac{H_T-h_L}{H_T}} \qquad (50)$$

The arrangement and design of a typical low-head siphon spillway is shown on figure 237. The figure includes an explanation of the design arrangement, a recommended design procedure, and approximate maximum values for the coefficients of discharge, C, for given assumptions of losses. The use of these discharge coefficients will provide satisfactory residual pressures at the crest of the throat.

G. STRUCTURAL DESIGN DETAILS

208. *General.*—The structural design of a spillway and the selection of specific structural details follow the determination of the spillway type and arrangement of components and the completion of the hydraulic design.

Usually the foundation material of a spillway is not competent to resist the destructive action of high-velocity flows; therefore, a non-erodible lining ordinarily must be provided along the spillway waterway. This lining may be of wood, steel,



TYPICAL LOW-HEAD SIPHON SPILLWAY

(Maximum head = atmospheric pressure equivalent of site)

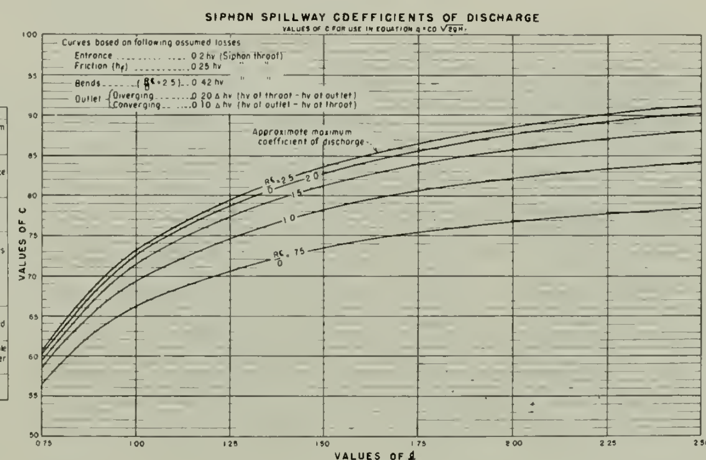
DESIGN DATA

$q = CD\sqrt{2gH_t} \leqq R_c\sqrt{0.7h_{at}(2g)} \times \log_e \frac{R_s}{R_c}$ = (Vortex flow—Max q)

q = Cubic ft per sec per ft of crest width
R_c = Radius of crest of throat in ft
R_s = Radius of summit of throat in ft
D = Throat size in ft.
d = Depth of water at outlet in ft
h_{at} = Atmospheric pressure at site in ft of water
H_t = Available head = (See outlet details)
C = Discharge coefficient based on $\frac{d}{D}$ and $\frac{R_c}{D}$
g = 32.16

SECTION	DESIGN	REASONS	REMARKS
Upper leg	A well proportioned transition providing a gradually contracting area. Inlet area should be 2 or 3 times the area of the throat	Provides accelerating velocity. Reduces entrance losses	Entrance loss for form shown suggested as 0.2 Δhv
Throat	Size depends upon required capacity. Recommended ratio of R_c to D is 2.5. For forming with timber D is 2'-0" min., metal forms left in place may be used for less than 2'-0". Recommended width of throat = 2D	Keeps bend losses low	Wide crests and shallow depths reduce priming time
Upper bend	Recommended ratio of R_c to D is 2.5. Aerating slot is set at 60° from crest. Pipe vent, set in sidewalls, connects slot to summit area	Keeps bend losses low. Slot prevents nappe from clinging to crest. Vent insures nappe trajectory	Area in upper and lower bend is same as area at throat
Lower bend	PC is set to contact nappe. PC is a point on trajectory of nappe as determined from desired priming head at crest. Recommended ratio of R_c to D is 2.5. Top of siphon at vertical section is set at elevation of outlet bottom. Air trap elevation is elevation of outlet bottom plus desired priming head. Vent area = throat area ÷ 48 (approx)	Provides seal and exhausts air from upper bend. Provides seal of lower bend and insures creation of partial vacuum. Vent exhausts entrained air with negligible loss of discharge	Vent decreases priming time, increases priming head slightly
Diverging tube	Outlet end set at desired elevation. Length of tube is set by 8°30' angle of divergence. Vent outlet set at $\frac{2}{3}$ depth (±) of outlet at max capacity	Provides transition, increases discharge	Discharge loss for form shown suggested as 0.2 Δhv
Siphon breaker	A pipe vent set at summit with inlet end set at or slightly below normal water surface. Area = throat area ÷ 24 (Min)	To break siphonic action slowly, prevents severe vibration	If practicable, adjustable inlet on siphon breaker is desirable
Seal at intake	Lip of upper leg is submerged an arbitrary distance, but should be set deep enough to prevent excessive draw down	Prevents "gulping" of air, and provides more normal action of siphon breaker	

Note: Design recommendations given above are presumed to give minimum losses according to experiments. Deviation therefrom should include consideration of additional losses.



REFERENCES

"Design of A High-Head Siphon Spillway" by Elmer Rock, A.S.C.E. Transactions, Vol 105, Page 1050
"A Study of Low-Head Siphons" by T.G. Owen—1939
U.S.B.R. Data on Model Tests, Wasteway No. 2 (Yakima Project) 1942
A.S.C.E. Transactions,—1934, Page 986
A.S.C.E. Transactions, 1939, Page 1789
"Siphon Spillways" by A.H. Naylor—1935

RECOMMENDED DESIGN PROCEDURE

1. Determine crest Elevation.
2. Determine head available (crest EL.—W.S. EL. at outlet)
3. Select preliminary values for D and R_c and solve for max q using Vortex flow formula $q = R_c\sqrt{0.7h_{at}(2g)}\log_e \frac{R_s}{R_c}$ (Recommend D = 2'-0" Min. and $\frac{R_c}{D}$ = 2.5)
4. Select values of d and H_t and obtain value of C from graph. Solve equation $q = CD\sqrt{2gH_t}$. (q must not be greater than value obtained in (3).)
5. Solve for required crest width = $\frac{Q}{q}$. Select crest width and number of siphons to be used for total Q and determine q for this crest width.
6. Repeat (3) and (4) until $q = CD\sqrt{2gH_t} \leqq R_c\sqrt{0.7h_{at}(2g)}\log_e \frac{R_s}{R_c}$
7. Select priming head by determining the reservoir W.S. EL. at which operation is desired. If priming head is less than $\frac{D}{3}$ locate PC of lower bend on trajectory of nappe.
8. Determine size and inlet EL. of siphon breaker (Area of siphon breaker = Area of siphon throat ÷ 24)
9. Select and design priming aids to be used (Slot to aerate nappe, Vent to equalize pressure above and below nappe, and vent at lower bend).

Figure 237. Typical low-head siphon spillway. From drawing 103–D–46.

handlaid grouted riprap, rubble masonry, or concrete. Such lining serves to prevent erosion, reduces friction losses by providing smooth bounding surfaces for the channel (which also permits smaller hydraulic sections), and provides a relatively watertight conveyance channel for directing flow past the dam. Economy and durability most often favor concrete as the appropriate lining material for water conveyance structures.

A spillway can be constructed on almost any foundation capable of sustaining applied loads without undue deformation. Although it is not usually advisable, a spillway can be placed on the face of an earthfill dam or through the dam, provided precautions are taken in the selection of design details to accommodate settlement and to prevent leakage from the structure. The type of walls, linings, and associated structures of a spillway and the details of the design will depend on the nature of the foundation. For instance, the details of the design for a spillway founded entirely in rock will differ from one constructed on clay. Structural details will differ according to foundation bearing capabilities, settlement or heave characteristics, and permeability and seepage features. Although concrete walls, linings, and associated structures may be adequate to withstand normal hydrostatic and earth loadings, they must also be arranged to allow for movements due to temperature change, and for unequal or large foundation settlements and heavings because of frost action. Provisions must also be made for handling leakage from the channel or underseepage from the foundation which might cause saturation of the underlying materials and large uplifts against the structure undersurfaces.

Subsequent sections discuss the structural designs of open channel spillways, including crest structures, walls, channel linings, and miscellaneous details. The structural designs of spillway conduits and tunnels are similar to those for outlet conduits and tunnels which are discussed in chapter IX.

209. *Crest Structures and Walls.* Spillway control structures and overflow crests against which reservoir heads act are essentially overflow dams, and spillway abutment structures or flanking dikes are similar to concrete nonoverflow dams or earthfill embankments. The design of earthfill dams is described in chapter V; the design of overflow and nonoverflow concrete dams is discussed in chapter VII.

The nature or type of confining side walls for open channel spillways will depend on the material upon which they are founded and on the loading to which they will be subjected. For spillway channels excavated in rock or firm material, and where sloping of the wall faces is permissible, lining placed directly against the excavated slopes may offer sufficient stability for forming the channel sidewalls. Otherwise, self-supporting retaining walls of the gravity, cantilever, or counterforted type will be required. Where the channel of a small spillway is relatively narrow and unequal settlement is not expected, the walls and floor can be made continuous as a monolithic flume-type section for increased stability and simplification of details. Where uplift under the structure is a consideration, as may be the case at a stilling basin, the flume-type structure will offer a more stable and secure structure.

The design of a gravity or reinforced concrete retaining wall is similar to that for a gravity dam, in that the stability against sliding and overturning and the magnitude and distribution of the foundation reaction resulting from the weight and applied loads must be determined. Methods of analyzing gravity structures for stability, including allowable sliding factors, and for determining foundation reactions are discussed in chapter VII. Suggested allowable bearing values are presented in appendix C.

Earth loadings are assumed on the basis of equivalent fluid pressures, based on cohesionless soil, as given by Rankine. Figure C-1 (app. C) gives criteria for determining earth loadings on vertical and inclined walls. Wall footings must be safeguarded against frost heaving, and the wall panels must be articulated to provide for adjustments in the event of foundation yielding or unequal settlement. To avoid differential settlement in soft or yielding foundations, wall footing dimensions should be selected to minimize foundation load concentrations and to provide nearly uniform bearing reactions across the base areas.

Inlet channel and chute walls may be subject to various combinations of loading. When flow is occurring through the spillway, hydrostatic loads on the outside of the wall tend to offset the loads caused by backfill. If, however, the fill has shrunk

away from the walls, the wall members may be subject to full outside waterload before the deflection is sufficient to gain support from the backfill. This condition is more likely to exist where the wall leans into the backfill. On the other hand, when the reservoir is drawn down below spillway level, there is no spill through the structure and the walls will be subject to full backfill loads without any support from waterloads. The structural design of wall members must recognize all these possibilities of loading. In the case of the assumption that the backfill may not be tight against the wall to help support the wall against water pressures, an increase in allowable stresses may be considered.

When permeable backfill is placed behind stilling basin walls or when the back of the wall is partly exposed to tailwater, the water pressure resulting from tailwater will need to be added to the backfill loading. For higher spillway discharges, the water level inside the basin will be depressed by the profile of the jump and an unbalanced hydrostatic load acting to overturn the walls will occur. The design loading assumptions must recognize this condition of unbalanced pressures as well as the increased uplift forces when sliding and overturning analyses are considered.

210. *Open Channel Linings.*—Floor pavings are provided primarily to form a reasonably watertight protective surfacing over the channel to prevent erosion or damage to the foundation. During spillway flows, the floor is subjected to hydrostatic forces due to the weight of the water in the channel, to boundary drag forces due to frictional resistance along the surface, to dynamic forces due to flow impingement, to uplift forces due to reduction of pressure along the boundary surface, or to uplift pressure caused by leakage through joints or cracks. When there are no spills, the floor is subject to the action of the elements including expansion and contraction due to temperature variations, alternate freezing and thawing, and weathering and chemical deterioration; to the effects of settlement and buckling; and to uplift pressures brought about by underseepage or high ground-water conditions. Since it is not always possible to evaluate the various forces which might occur nor to make the lining heavy enough to resist them, the thickness of the lining is most often established on a more or less arbitrary basis; and underdrains, anchors, cutoffs, etc., are utilized to stabilize the floor.

To provide a relatively watertight lining which will withstand reasonable weathering and abrasion, and which will hold up against ordinarily experienced forces caused by expansion, contraction, frost heave, and settlement of the foundation, a nominal minimum thickness of 8 inches is recommended for small spillways when the lining is placed directly on rock. When the lining is placed on earth or on an intervening gravel layer, a somewhat thicker slab should be provided to forestall cracking or buckling if expansion and contraction can move or displace the slab.

When a spillway channel is excavated in rock, the paving slab is cast directly on the excavated surface. Anchor bars grouted into holes drilled into the rock can be provided to tie the paving to the foundation. A slab which is bonded to the foundation may not move as the result of expansion and contraction. Instead, numerous cracks which in effect divide the slab into a series of small individual blocks will occur. Reinforcement therefore must be provided to tie these individual blocks together and to distribute the cracking and minimize the crack openings. Typical details for paving slabs on rock are shown on figure 238(A). The anchorage provided increases the effective weight of the slab against displacement by the amount of foundation rock to which the anchors can be tied. Depth and spacing of anchors will depend on the nature of the bedrock, its stratification, jointing, weathering, etc. The anchor should be of sufficient size to hold the weight of the foundation to which it is anchored without exceeding the yield stress of the steel. A gridwork of underdrains laid with open joints in gravel-filled trenches is provided to prevent a buildup of uplift under the paving. When leakage through the joints is to be minimized, metal or rubber waterstops are provided.

When a spillway channel is excavated through earth, the paving slab may be cast directly on the excavated surface, or an intervening pervious blanket may be required, depending on the nature of the foundation as related to its permeability, susceptibility to heaving from frost action, and heterogeneity as it may affect differential settlement. Because the slab is not bonded to the foundation, it is subject to movement from ex-

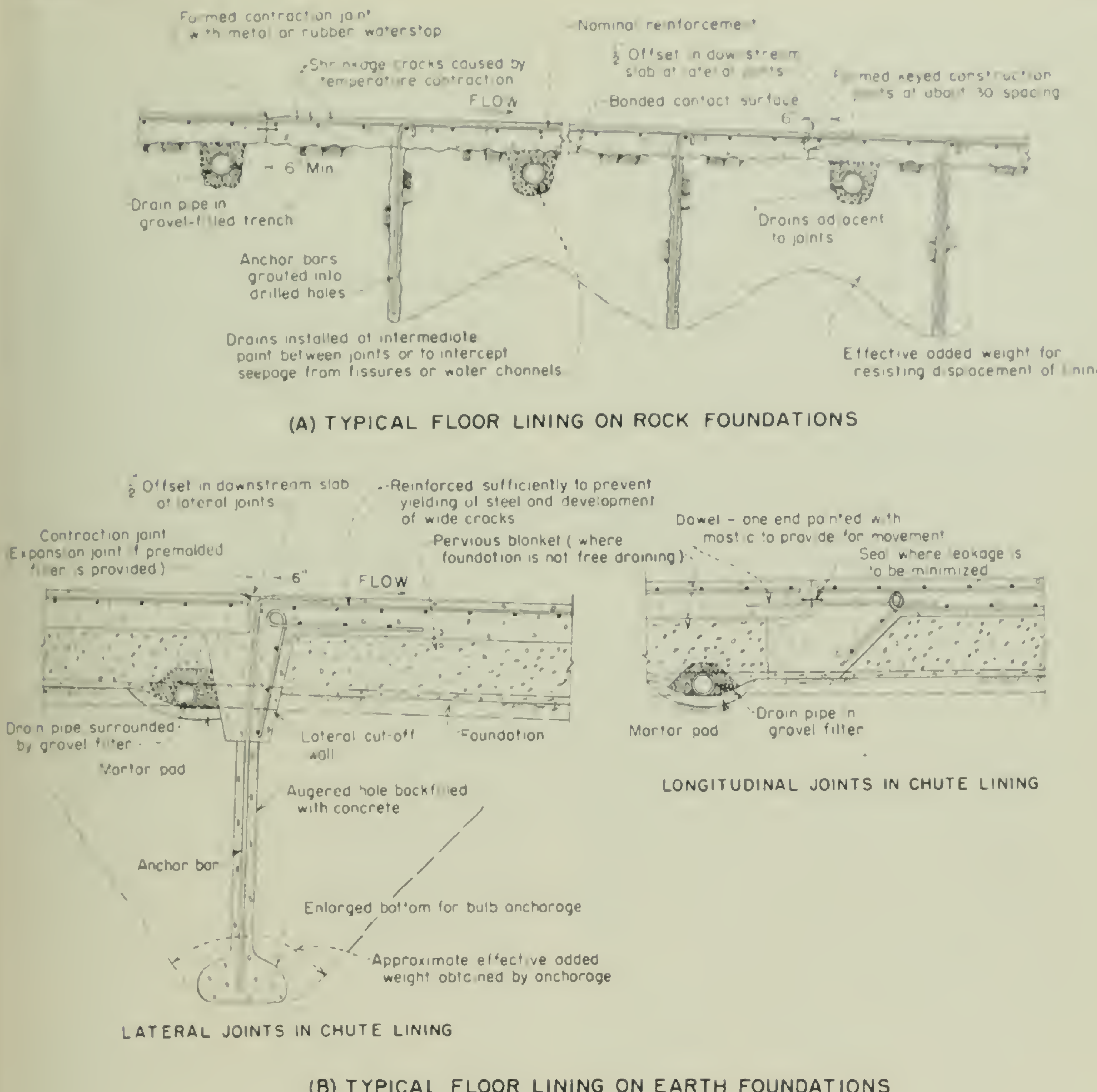


Figure 238. **Floor lining details for spillway channels.**

pansion or contraction, and it must be restrained from creeping when it is constructed on a slope. This restraint is best achieved by cutoffs which can be held in a more or less fixed position with respect to the slab and to the foundation, or by tieing the slab to walls, piles, or similar rigid members of the spillway structure. Since the slab is relatively free to move upon the foundation, the movement will take place from the fixed edges and the paving should be reinforced sufficiently to permit its sliding without cracking of the concrete or yielding of the reinforcement. To assist further in holding the slab to the foundation, bulb anchors are sometimes employed as shown on figure 238(B). The anchor in this instance in effect ties the slab to a cone of earth, the volume of which will depend on the anchor depth and spacing and on the angle of internal friction of the soil.

A pervious gravel blanket is often provided between the slab and the foundation when the

foundation is sufficiently impervious to prevent leakage from draining away, or where it is subject to capillarity which will draw moisture to the underside of the lining. The blanket serves not only as a free-draining medium but also aids in insulating the foundation from frost penetration. The thickness of the blanket thus depends on the climate at the site and on the susceptibility of the foundation to frost heaving. A gridwork of underdrains laid with open joints in gravel and bedded on a mortar pad to prevent the foundation material from being leached into the pipe is provided as a collecting system for the seepage. The network of drainage pipe empties into one or more trunk drains which carry the seepage flows to outlets through the channel floor or walls.

In stratified foundations, ground water or seepage can cause uplift on layers below the floor lining, and drainage holes are sometimes augered into the underlying material and backfilled with gravels to relieve the underpressure.

When watertightness of the paving against exterior water heads is required, metal or rubber waterstops are installed to seal the joints. Such seals are provided in floor slabs upstream from the control structure if watertightness is desired to increase the percolation path under the structure. They are commonly provided at transverse joints along concave curved portions of the downstream channel where the dynamic pressures on the floor cause a high head for introducing water into the joint. Seals may be desirable along longitudinal joints in a stilling basin on a permeable base. Differential heads resulting from the sloping water surface of the jump can cause a circulating flow under the slab if leakage is allowed to enter the joint at the downstream end of the basin and to flow out of the joint at the upstream end.

Lateral joints over which flow velocities are high are arranged so that the upstream edge of the lower slab cannot heave without moving the lower edge of the upper slab a like amount; further, the lower slab edge is constructed about one-half inch lower than the upper slab edge. These provisions are made to forestall a high buildup of dynamic head at the joint which would result if the surface of the downstream slab were to project above the surface of the upstream slab. The dynamic head could introduce water at high pressure under the slab, which would result in uplift or dislodgement.

Contraction joints are generally placed from 25 to 50 feet apart in both the floor and walls. Joints are also provided where angular changes of the floor surface occur and where they are required to avoid reentrant angles in the slab which often cause cracking of the paving. The use of joint fillers in contraction joints should be minimized because deterioration of the filler will result in an open joint which is difficult to maintain. If joints are provided at the indicated spacings, the contraction or expansion movements may not be severe and filler material in the joint may not be necessary. Floor slabs can be constructed in alternate panels; the initial placement shrinkage of the concrete may then afford sufficient joint opening for subsequent expansion. Keyed joints in thin floors and walls which may be subjected to differential movement are unsatisfactory, since inequalities in deflection across the joint will place high stress on the keys or keyways and cause them to spall. An unkeyed joint with slip dowels is a better detail.

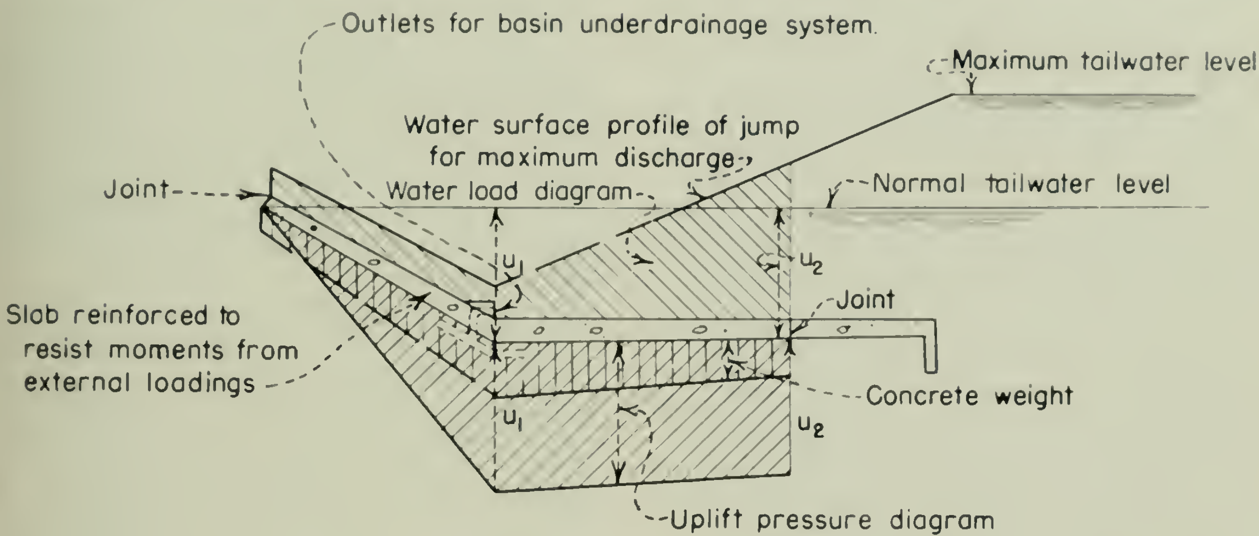
Water normally will stand in a stilling basin whose floor is at a lower level than the river channel. With this condition the foundation under the basin will be permanently saturated. When the water is lowered in the basin, the floor can be subjected to an uplift equal to the tailwater head or higher if the pressure is augmented by head from a higher source. During times of spillway discharge, the water weight in the basin will be reduced because of the slope of the jump profile, and at the upstream end of the basin the uplift will far exceed the downward weight. The basin floor must be heavy enough to withstand this unbalanced waterload, unless an adequate drainage system is installed to relieve the uplift pressure when jump sweep-out occurs. Since a drainage system cannot be considered entirely effective because of the possibility of clogging or silting of the drain outlets, the floor slab is usually made sufficiently heavy to resist the flotation effect on the floor.

For design, the stilling basin floor is considered to be a free body in static equilibrium with foundation reactions balancing active loads. Uplift forces caused by hydrostatic head on the bottom of the slab are counterbalanced by the weight of

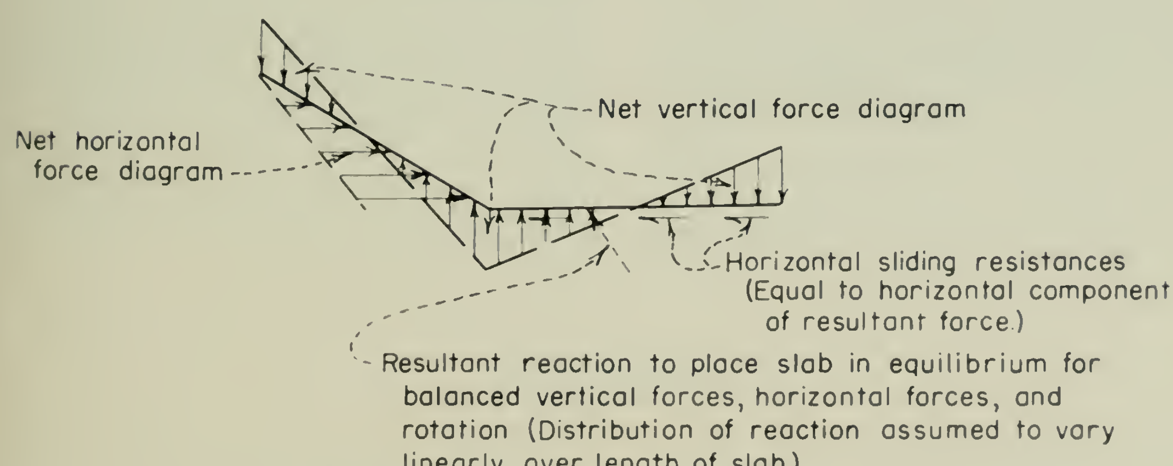
the concrete and the effective weight of the water in the basin. Differential horizontal hydrostatic forces are opposed by the sliding resistance of the horizontal leg of the slab on the foundation. Equilibrium against rotation is achieved by equating any unbalanced forces with a foundation reaction force positioned so that the moments will be zero about any point.

An illustrative force diagram for a typical basin analysis is shown on figure 239. Note that the dynamic force which will occur because of impingement of the incoming high-velocity flow on the horizontal apron is not included. Where chute blocks or dentates are used at the upstream end of the apron, reduced pressure zones opposite the blocks will occur. Because the reduced pressures and the impingement forces tend to offset each other, they are both neglected in the analysis.

It will be seen that the weight required in the floor will vary in direct relation to the amount of assumed uplift. If uplift equal to the head from maximum tailwater is considered, the required floor thickness will be almost prohibitive. Therefore, an uplift based on a lesser tailwater level,



(A) LOADINGS ON FLOOR



(B) FORCE DIAGRAMS

Figure 239. Illustration of uplift forces acting on a stilling basin floor.

such as that from the normal tailwater condition, is generally assumed and reliance is placed on the drainage system for relief of greater uplift pressures. Ordinarily the outlets for the basin underdrainage system are located in the sills near the upstream end of the basin floor. The head on these outlets will be reduced when the jump occurs, which facilitates relief of hydrostatic pressures under the slab. The stilling basin floor slab must be designed to withstand the internal moments resulting from the external loadings indicated by the force diagram. The slab will be thickest at the junction of the sloping leg and the horizontal apron. Reinforcement will be required in the slab to withstand the computed internal stresses.

211. *Miscellaneous Details.*—(a) *Cutoffs.*—One or more cutoffs are generally provided at the upstream end of a spillway for various purposes. They may form a watertight curtain against seepage under the structure, or they may be used to increase the path of percolation under the structure and thus reduce uplift forces. Cutoffs also can be used to intercept permeable strata in the foundation so as to minimize seepage and prevent a buildup of uplift pressure under the spillway or adjacent areas. When the cutoff trench for the dam extends to the spillway, it is generally joined to the upstream spillway cutoff to provide a continuous barrier across the abutment area. In jointed rock the cutoff acts as a grout cap for a grout curtain which is often extended across the spillway foundation.

A cutoff is usually provided at the downstream end of a spillway structure as a safeguard against erosion and undermining of the end of the structure. Cutoffs at intermediate points along the length of a spillway are sometimes provided as barriers against water flowing along the contact between the structure and the foundation and to lengthen the path of percolation under the structure. When the spillway is a conduit under the dam, the cutoff takes the shape of collars placed at intervals around the conduit barrel. Wherever possible, cutoffs in rock foundations are placed in vertical trenches. In earth foundations where the cutoffs must be formed in a trench with sloping sides, care must be taken to compact the trench backfill properly with impervious material to obtain a reasonably watertight barrier.

(b) *Backfill.*—When a spillway is placed adjacent to a dam so that the impervious zone of the embankment abuts the spillway walls, the wall backfill is actually the impervious zone of the dam and is similarly compacted. Backfill elsewhere along the spillway walls ordinarily should be free-draining material to minimize hydrostatic pressures against the walls. Backfill other than that adjacent to the dam may be either compacted or uncompacted. The choice of backfill material and the compaction methods used in placing such material will affect the design loadings on the walls.

(c) *Riprap.*—When the spillway approach channel is excavated in material that will be eroded by the approach velocities, a zone of riprap is often provided immediately upstream from the inlet lining to prevent scour of the channel floor and side slopes adjacent to the spillway concrete. The riprap is generally a continuation of that along the upstream face of the dam, is of similar size and gradation, and has similar bedding. Riprap is normally used in the outlet channel adjacent to the downstream cutoff to prevent excessive erosion and undermining of the downstream end of the structure. To resist scour from high exit velocities, the riprap should be the largest size possible and should be bedded on a graded material. The voids should be filled with spalls to prevent the underlying material from washing out, which would cause the riprap to settle or to be displaced.

H. BIBLIOGRAPHY

212. *Bibliography.*

[1] Donnelly, C. A., and Blaisdell, F. W., "Straight Drop Spillway Stilling Basin," University of Minnesota, Saint Anthony Falls Hydraulic Laboratory, Technical Paper No. 15, series B, November 1954.

[2] Morris, B. T., and Johnson, D. C., Hydraulic Design of Drop Structures for Gully Control," Trans. ASCE, vol. 108, 1943, p. 887.

[3] Moore, W. L., "Energy Loss at the Base of a Free Overfall," Trans. ASCE, vol. 108, 1943, p. 1343.

[4] Rand, Walter, "Flow Geometry at Straight Drop Spillways," ASCE Proceedings, Paper No. 791, September 1955.

[5] Peterka, A. J., "Morning-Glory Shaft Spillways," Trans. ASCE, vol. 121, 1956, p. 385.

[6] Bradley, J. N., "Morning-Glory Shaft Spillways: Prototype Behavior," Trans. ASCE, vol. 121, 1956, p. 312.

[7] Bureau of Reclamation, "Studies of Crests of Overfall Dams," Bulletin 3, Part VI, Hydraulic Investigations, Boulder Canyon Project Final Reports, 1948.

[8] Hinds, Julian, "Side Channel Spillways," Trans. ASCE, vol. 89, 1926, p. 881.

[9] Bradley, J. N., and Peterka, A. J., "The Hydraulic Design of Stilling Basins," ASCE Proceedings, vol. 83, October 1957, Journal of the Hydraulics Division, No. HY5, Papers No. 1401 to 1406, inclusive.

[10] Warnock, J. E., "Experiments Aid in Design at Grand Coulee," Civil Engineering, vol. 6, 1936, p. 737.

[11] Bureau of Reclamation, "Hydraulic Design of Stilling Basin and Bucket Energy Dissipators," Engineering Monograph No. 25, 1958.

[12] Bradley, J. N., and Peterka, A. J., "Hydraulic Design of Stilling Basins: Small Basins for Pipe or Open Channel Outlets—No Tailwater Required (Basin VI)," ASCE Proceedings, vol. 83, October 1957, Journal of the Hydraulics Division, No. HY5, Paper No. 1406.

[13] Doddiah, D., Albertson, M. L., and Thomas, R. A., "Scour From Jets," Proceedings, Minnesota International Hydraulics Convention (Joint Meeting of International Association for Hydraulic Research and Hydraulics Division, ASCE), Minneapolis, Minn., August 1953, p. 161.

[14] Scimemi, Ettore, "Discussion of Paper 'Model Study of Brown Canyon Debris Barrier' by Bermeal and Sanks," Trans. SACE, vol. 112, 1947, p. 1016.

[15] Wagner, W. E., "Morning Glory Shaft Spillways: Determination of Pressure-Controlled Profiles," Trans. ASCE, vol. 121, 1956, p. 345.

[16] Mavis, F. T., "The Hydraulics of Culverts," Pennsylvania State College Engineering Experiment Station, Bulletin 56, vol. XXXVII, No. 7, February 12, 1943.

[17] Straub, L. G., Anderson, A. G., and Bowers, C. E., "Importance of Inlet Design on Culvert Capacity, Culvert Hydraulics," Highway Research Board of the National Academy of Sciences, National Research Council, Publication 287, Research Report No. 15-B, p. 53, 1953.

[18] Karr, M. H., and Clayton, L. A., "Model Studies of Inlet Designs for Pipe Culverts on Steep Grades," Engineering Experiment Station, Oregon State College, Bulletin No. 35, June 1954.

[19] Schiller, R. E. Jr., "Tests on Circular Pipe Culvert Inlets," Culvert Flow Characteristics, Highway Research Board of the National Academy of Sciences, National Research Council, Publication 413, Bulletin No. 126, p. 11, 1956.

[20] Shoemaker, R. H. Jr., and Clayton, L. A., "Model Studies of Tapered Inlets for Box Culverts," Culvert Hydraulics, Highway Research Board of the National Academy of Sciences, National Research Council, Publication 287, Research Report No. 15-B, p. 1.

[21] Blaisdell, F. W., "Hydraulics of Closed Conduit Spillways—Part I—Theory and Its Application," University of Minnesota, Saint Anthony Falls Hydraulic Laboratory, Technical Paper No. 12, series B, January 1952, revised February 1958.

[22] Blaisdell, F. W., "Hydraulics of Closed Conduit Spillways—Parts II through VII—Results of Tests on Several Forms of the Spillway," University of Minnesota, Saint Anthony Falls Hydraulic Laboratory, Technical Paper No. 18, series B, March 1958.

[23] Blaisdell, F. W., "Hydraulics of Closed Conduit Spillways—Part VIII—Miscellaneous Laboratory Tests and Part IX—Field Tests," University of Minnesota, Saint Anthony Falls Hydraulic Laboratory, Technical Paper No. 19, series B, March 1958.

[24] Blaisdell, F. W., and Donnelly, C. A., "Hydraulics of Closed Conduit Spillways—Part X—The Hood Inlet, University of Minnesota, Saint Anthony Falls Hydraulic Laboratory, Technical Paper No. 20, series B, April 1958.

Outlet Works

C. J. HOFFMAN [1]

A. GENERAL

213. ***Functions of Outlet Works.*** An outlet works serves to regulate or release water impounded by a dam. It may release incoming flows at a retarded rate, as in the case of a detention dam; divert incoming flows into canals or pipelines, as in the case of a diversion dam; or release stored waters at such rates as may be dictated by downstream needs, evacuation considerations, or a combination of multiple-purpose requirements.

Outlet works structures can be classified according to their purpose, their physical and structural arrangement, or their hydraulic operation. An outlet works which empties directly into the river could be designated a river outlet; one which discharges into a canal could be classed as a canal outlet; and one which delivers water into a closed pipe system could be termed a pressure pipe outlet. An outlet works may be described according to whether it consists of an open channel or closed conduit waterway, or whether the closed waterway is a conduit in cut-and-cover or in tunnel. The outlet works may also be classified according to its hydraulic operation, whether it is gated or ungated or, for a closed conduit, whether it flows under pressure for part or all of its length or only as a free-flow waterway. Typical outlet works installations are shown on figures 240 through 246.

Occasionally the outlet may be placed at a higher level to deliver water to a canal, and a bypass extended to the river to furnish necessary flows below the dam. Such flows may be required to satisfy prior right uses downstream from the site; or they may be required for the maintenance of a live stream for abatement of stream pollution, preservation of aquatic life, or stock watering purposes. For dams constructed to provide reservoirs principally for recreation or fish and wildlife, a fairly constant lake level is desired and an outlet works may be needed only to release the minimum flows which will provide a live stream below the dam.

In certain instances the outlet works of a dam may be used in lieu of a service spillway in conjunction with an auxiliary or secondary spillway. In this event the usual outlet works installation might be modified to include a bypass overflow, so that the structure can serve as both an outlet works and a spillway. Such structures are typified by figures 154 and 246, for Wasco Dam and Lion Lake dikes, respectively. In these installations the overflow weirs in the control shaft automatically bypass surplus inflows whenever the reservoir rises above normal storage level.

An outlet works may also act as a flood control regulator, to release waters temporarily stored in flood control storage space or to evacuate storage in anticipation of flood inflows. Further, the outlets may serve to empty the reservoir to permit inspection, to make needed repairs, or to maintain the upstream face of the dam or other structures normally inundated. The outlets may also aid in lowering the reservoir storage when it is desired to control or to poison scrap fish or other objectionable aquatic life in the reservoir.

214. ***Determination of Required Capacities.*** Outlet works controls are designed to release water at specific rates, as dictated by downstream needs, flood control regulation, storage considerations, or legal requirements. Delivery of irrigation water is usually determined from project or farm needs and is related to the consumptive use and

[1] Engineer, Spillways and Outlet Works Section, Bureau of Reclamation.

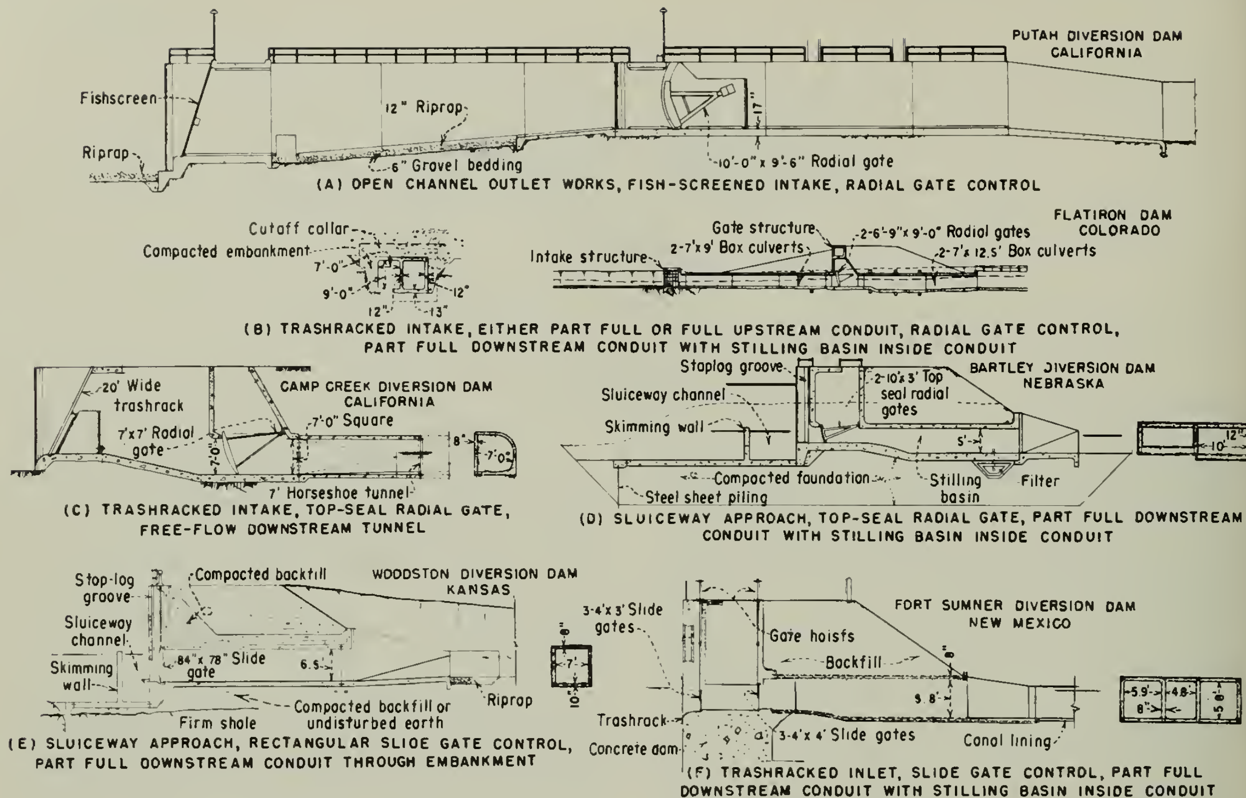


Figure 240. Typical low-head outlet works installations.

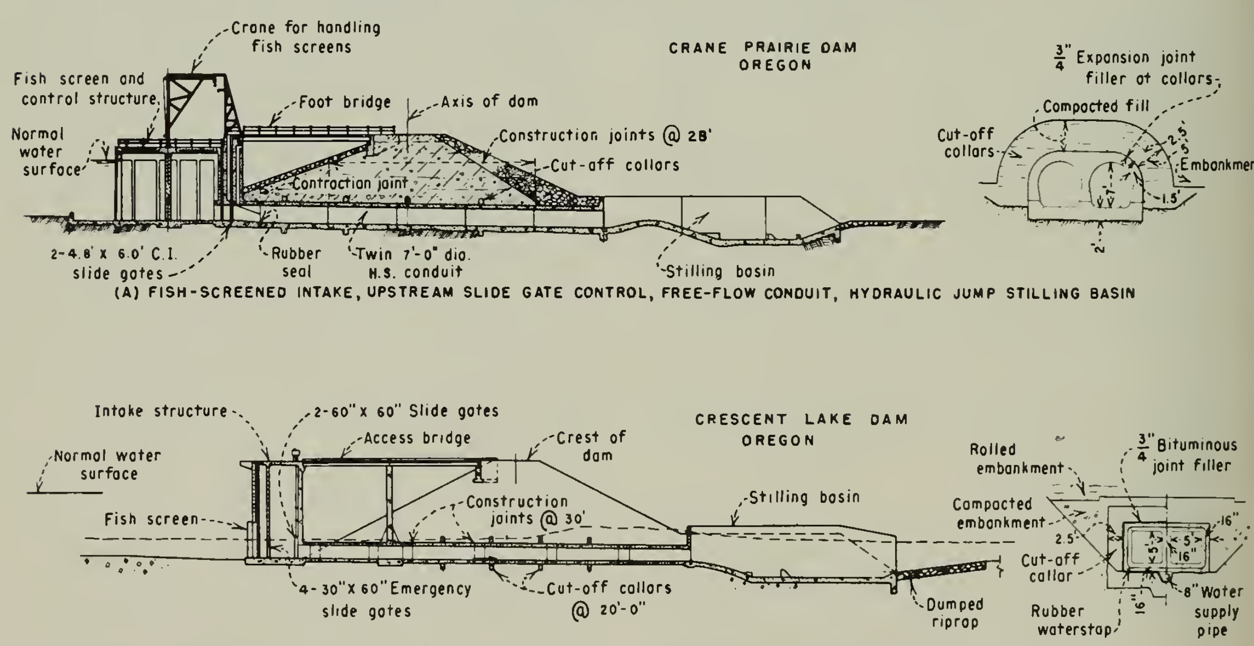


Figure 241. Typical free-flow conduit outlet works installations.

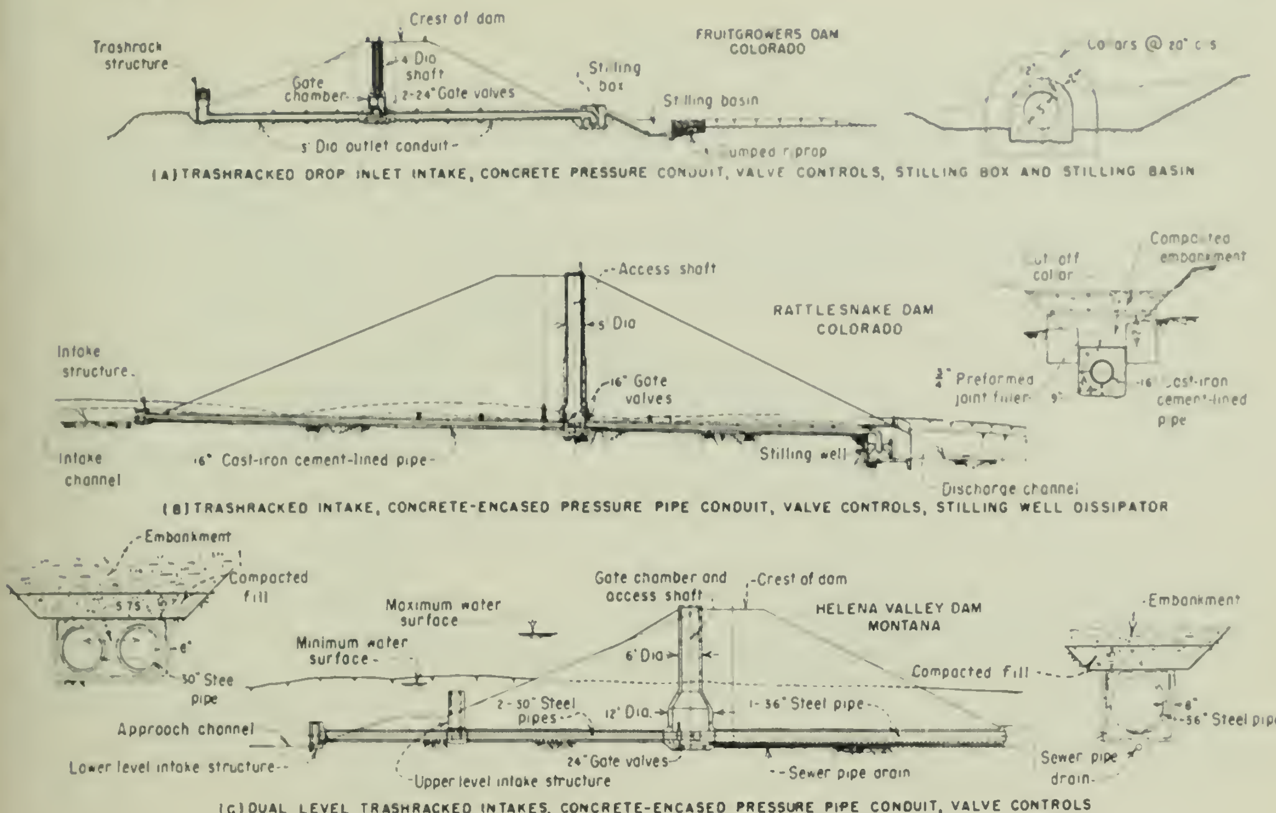


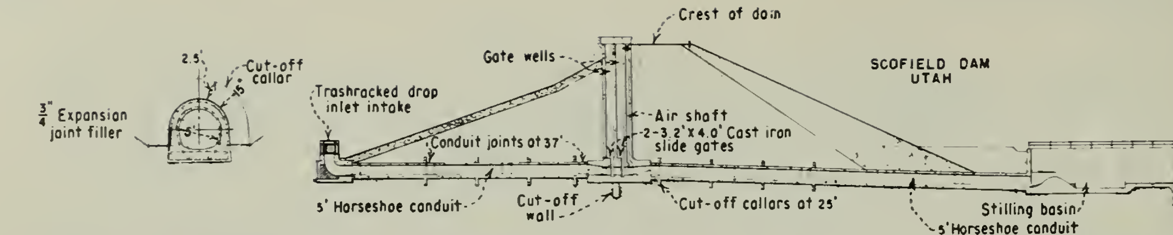
Figure 242. **Typical pressure conduit outlet works installations.**

to any special water requirements of the irrigation system. Delivery for domestic use can be similarly established. Releases of flows to satisfy prior rights must generally be included with other needed releases. Minimum downstream flows for pollution abatement, fish preservation, and other companion needs may often be accommodated through other required releases.

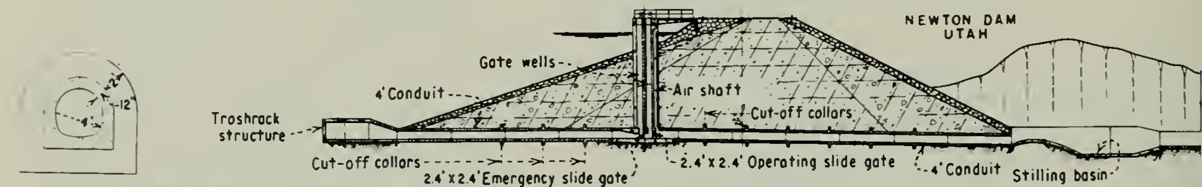
Irrigation outlet capacities are determined from reservoir operation studies and must be based on a consideration of a critical period of low runoff when reservoir storages are low and daily irrigation demands are at their peak. The most critical draft from the reservoir, considering such demands (commensurate with remaining reservoir storage) together with prior rights or other needed releases, generally determines the minimum irrigation outlet capacity. These requirements are stated in terms of discharge at either a given reservoir content or water surface elevation. Occasionally outlet capacity requirements are established for several reservoir contents or alternate water surfaces. For example, outlet requirements may be set forth as: 20 second-feet capacity at reservoir content 500 acre-feet, and 100 second-feet capacity at reservoir content 3,000 acre-feet.

Evacuation of waters stored in an allocated flood control storage space of a reservoir can be accomplished through a gated spillway at the higher reservoir levels or through an outlet at the lower levels. Flood control releases generally can be combined with the irrigation outlet releases if the outlet empties into a river instead of into a canal. The capacity of the flood control outlet is determined by the required time of evacuation of a given storage space, considering the inflow into the reservoir during this emptying period. The combined flood control and irrigation releases ordinarily must not exceed the safe channel capacity of the river downstream from the dam and must allow for any anticipated inflows immediately below the dam. These inflows may be the natural runoffs or may result from releases from other storage developments along the river or from adjacent developments on tributaries emptying into the river.

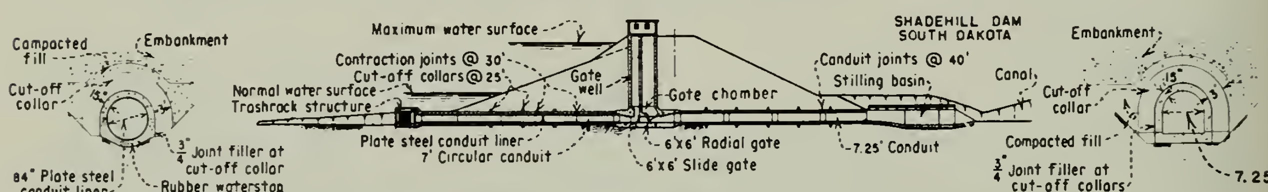
If an outlet is to serve as a service spillway in



(A) TRASHRACKED DROP INLET INTAKE, PRESSURE UPSTREAM CONDUIT, SLIDE GATE CONTROL WITH WET WELL SHAFT, FREE-FLOW DOWNSTREAM CONDUIT, STILLING BASIN

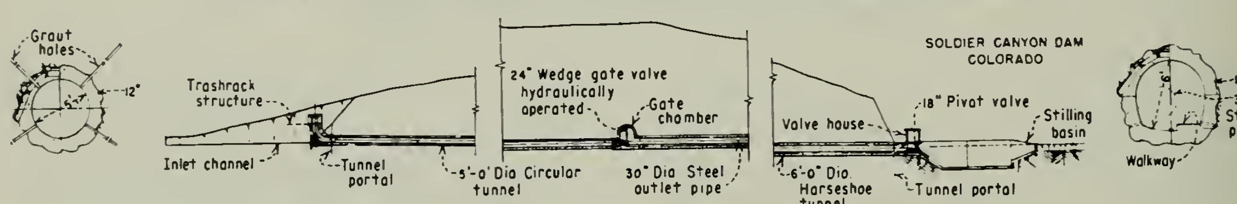


(B) TRASHRACKED BOX INTAKE, PRESSURE UPSTREAM CONDUIT, SLIDE GATE CONTROLS IN WET WELL SHAFT, FREE-FLOW DOWNSTREAM CONDUIT, STILLING BASIN

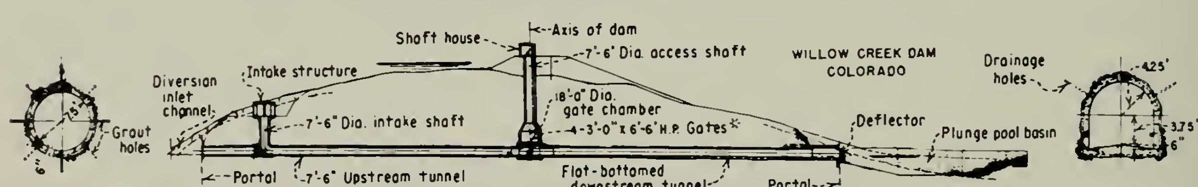


(C) TRASHRACKED BOX INTAKE, STEEL-LINED PRESSURE UPSTREAM CONDUIT, SLIDE EMERGENCY GATE IN WET WELL, RADIAL OPERATING GATE, FREE-FLOW DOWNSTREAM CONDUIT, STILLING BASIN

Figure 243. **Typical combined pressure and free-flow conduit outlet works installations.**

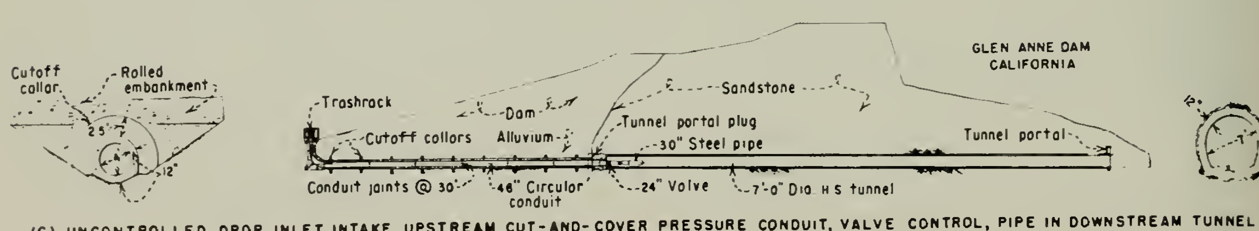


(A) UNCONTROLLED DROP INLET INTAKE. UPSTREAM PRESSURE TUNNEL, PIPE IN DOWNSTREAM TUNNEL, VALVE CONTROL AT OUTLET, STILLING BASIN



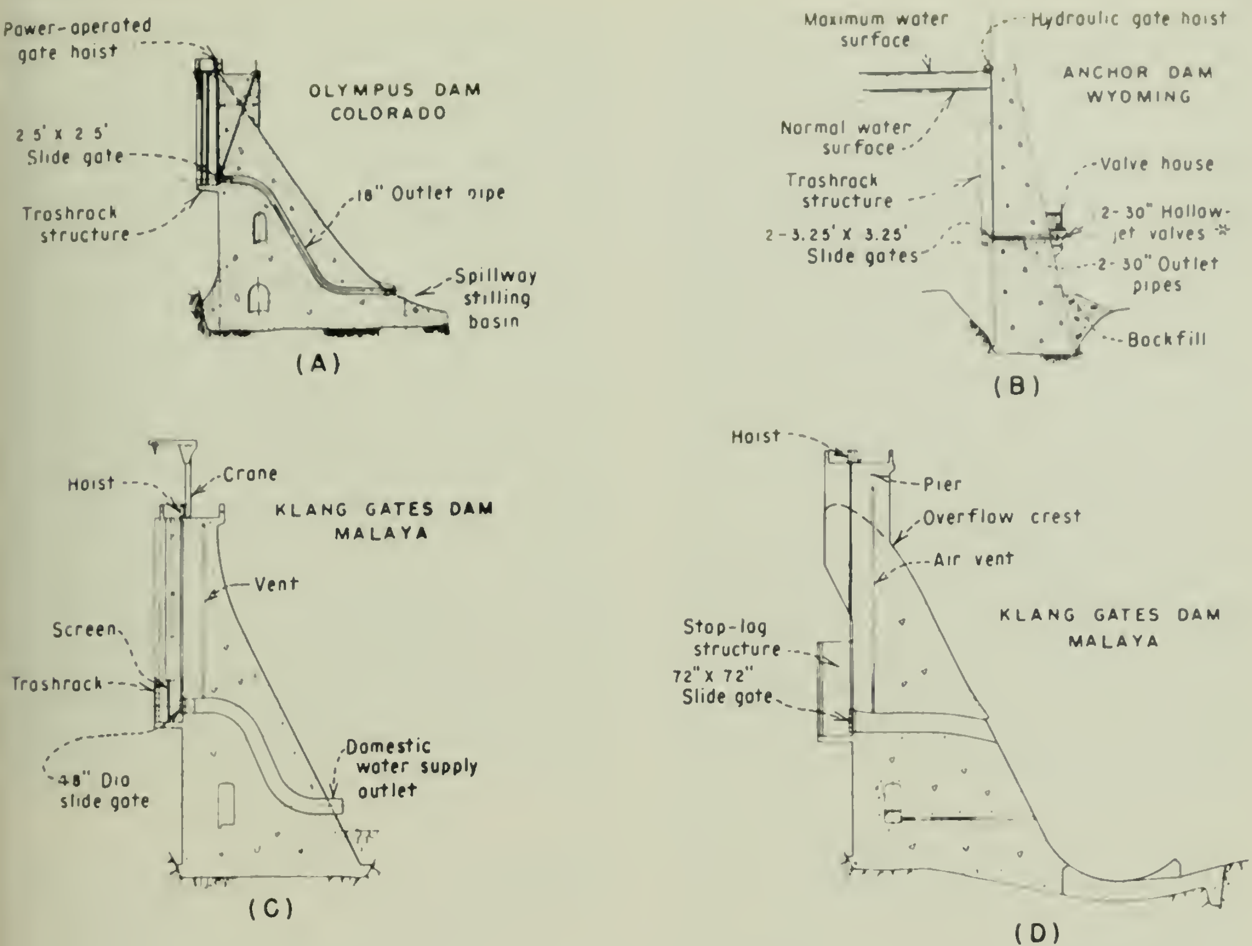
(B) UNCONTROLLED DROP INLET INTAKE IN SHAFT, UPSTREAM PRESSURE TUNNEL, SLIDE GATE CONTROL, FREE-FLOW DOWNSTREAM TUNNEL, DEFLECTOR OUTLET, PLUNGE POOL BASIN

(✲) 4-4'-0"x 5'-0" Bonneted slide gates can be substituted for low-head installations)



(C) UNCONTROLLED DROP INLET INTAKE, UPSTREAM CUT-AND-COVER PRESSURE CONDUIT, VALVE CONTROL, PIPE IN DOWNSTREAM TUNNEL

Figure 244. **Typical tunnel outlet works installations.**



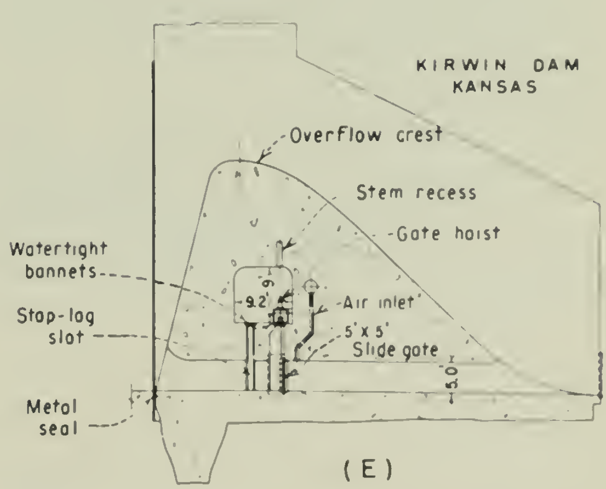
(A) Sluice through non-overflow section emptying into spillway stilling basin. Trashracked intake, slide gate control at upstream face of dam. Free-flow pipe

(B) Outlet pipe through non-overflow section. Trashracked intake, upstream emergency slide gate. Downstream valve control, freely discharging.

(C) Outlet pipe through non-overflow section. Trashracked intake, upstream slide gate control. Pressure pipe.

(D) Sluice through spillway section. Upstream slide gate control, downstream free-flow conduit

(E) Sluice through spillway section of dam controlled by slide gate Gate operated from gallery in dam. Upstream pressure conduit, downstream free-flow conduit.



(* Butterfly valves can be substituted for low-head installations)

Figure 245. Typical outlet works installations for concrete dams.

releasing surplus inflows from the reservoir, the required discharge for this purpose may fix the outlet capacity. Similarly, for emptying the reservoir for inspection or repair, the volume of water to be evacuated and the allotted emptying period may be the determining conditions for establishing the minimum outlet capacity. Here again, the inflow into the reservoir during the emptying period must be considered. The capacity at low reservoir level should be at least equal to the average inflow expected during the maintenance or repair period. It can, of course, be assumed that any required repair work might be delayed until service demands are light and that it will be done at times of low inflow and at seasons favorable to such construction.

An outlet works cut-and-cover conduit or tunnel often may be utilized for diverting the riverflow during the construction period, thus avoiding the necessity for supplementary installations for that purpose. The outlet structure size dictated by this use rather than the size indicated for ordinary outlet works requirements may determine the final outlet works capacity.

215. *Outlet Works Position in Relation to Reservoir Storage Levels.*—The establishment of the intake level and the elevations of the outlet controls and the conveyance passageway, as they relate to the reservoir storage levels, are influenced by many considerations. Primarily, in order to attain the required discharge capacity, the outlet must be placed sufficiently below minimum reservoir operating level to provide head for effecting outlet works flows.

Outlet works for small detention dams are generally constructed near riverbed level since permanent storage space, except for silt retention, is ordinarily not provided. (These outlet works may be ungated in order to retard the outflow while the reservoir temporarily stores the bulk of the flood runoff, or they may be gated in order to regulate the releases of the temporarily stored waters.) If the purpose of the dam is only to raise and divert incoming flows, the main outlet works generally is a headworks or regulating structure at a high level, and a sluiceway or small bypass outlet is provided to furnish water to the river downstream or to drain the water from behind the

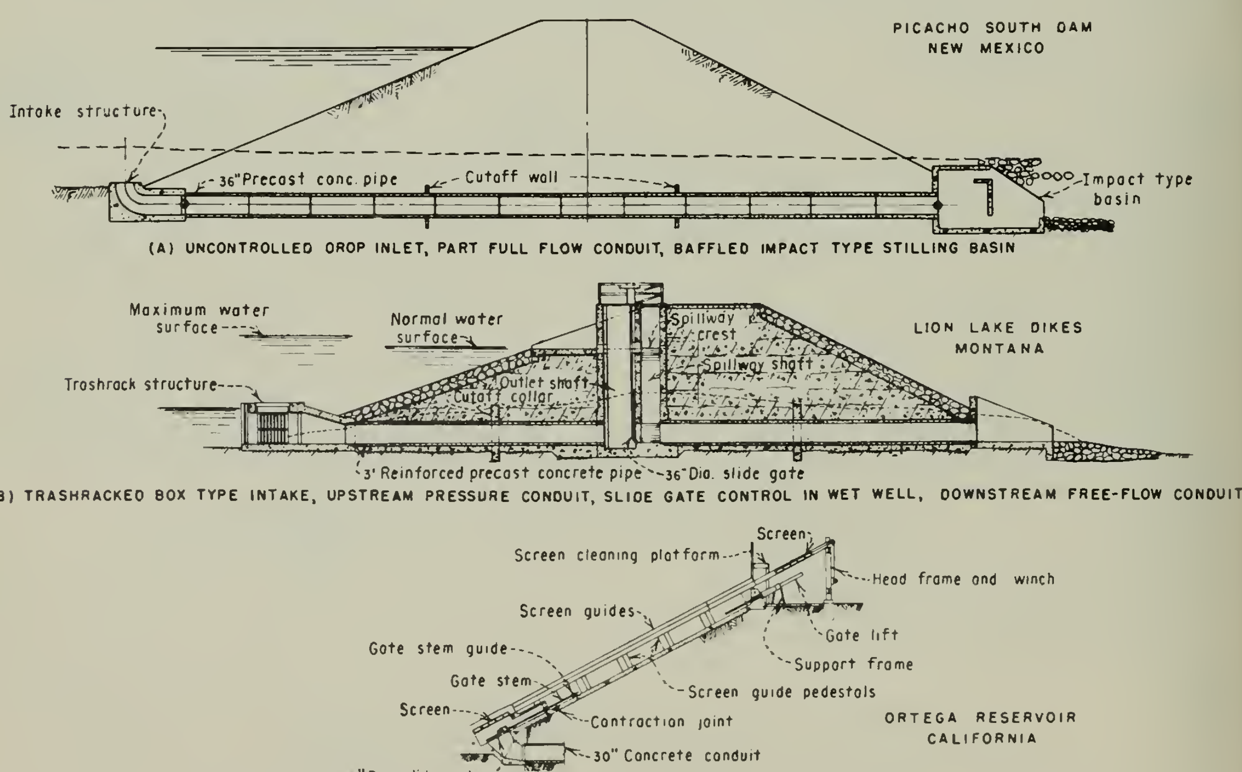


Figure 246. Typical precast pipe outlet works installations.

dam during off-season periods. For dams which impound waters for irrigation, domestic use, or other conservation purposes, the outlet works must be placed low enough to draw the reservoir down to the bottom of the allocated storage space; however, it might be placed at some level above the riverbed, depending on the elevation of the established minimum reservoir storage level.

It is usual practice to make an allowance in the bottom of a storage reservoir for inactive storage for sediment deposition, fish and wildlife, and recreation. The positioning of the intake sill then becomes an important consideration, since it must be high enough to prevent interference from the sediment deposits, but at the same time low enough to permit either a partial or a complete drawdown below the top of the inactive storage.

As is discussed in section 227, the size of an outlet conduit for a required discharge varies according to an inverse relationship with the available head for producing the discharge. This relationship may be expressed by the following equation:

$$H_T=K_1 h_v, \text{ or } H_T=K_2\frac{Q^2}{a^2},$$

where:

H_T= the total available head for producing flow,
Q= the required outlet works discharge, and
a = the required area of the conduit.

The above relationship for a particular design is illustrated on figure 247(A). In this example, if the head available for the required outlet works discharge is increased from 1.6 to 4.6 feet, the corresponding conduit diameter can be decreased from 6 to 4.75 feet. This shows that the conduit size can be significantly reduced if the inactive storage level can be increased. The reduction in active storage capacity resulting from increasing the inactive storage level 3 feet would have to be compensated by adding an equivalent amount of capacity to the top of the pool. By referring to the reservoir capacity curve, figure 247(B), it will be apparent that for equivalent storages (represented by *de* and *gh*) the 3 feet of head represented by ordinate *cd* added to obtain a reduced outlet works size would require a much smaller increase (represented by the ordinate *fg*) in the height of the dam. Thus, economic studies can be utilized to determine the proper outlet size in relation to the minimum reservoir storage level.

Where an outlet is placed at riverbed level to accommodate the construction diversion plan or to drain the reservoir, the operating sill can be placed at a higher level to provide a sediment and debris basin and other desired inactive storage space, or the intake can be designed to permit raising the sill as sediment accumulates. During the construction period, a temporary diversion opening can be formed in the base of the intake for handling diversion flows and later closed with a plug. For emptying the reservoir, a bypass can be installed around the intake at riverbed level, either emptying into the lower portion of the conduit or passing under it. Delivery of water to a canal at a higher level can be made by a pressure riser pipe connecting the conduit to the canal.

216. *Conditions Which Determine Outlet Works Layout.* The layout of a particular outlet works will be influenced by many conditions relating to the hydraulic requirements, to the site adaptability and the interrelation of the outlet works to the construction procedures, and to other appurtenances of the development. Thus, an outlet works leading to a high-level canal or into a closed pipeline might differ from one emptying into the river. Similarly, a scheme in which the outlet works is used for diversion might vary from one where diversion is effected by other means. In certain instances, the proximity of the spillway may permit combining some of the outlet works and spillway components into a single structure. As an example, the spillway and outlet works layout might be arranged so that discharges from both structures will empty into a common stilling basin. An interesting arrangement in which a spillway and outlet works are combined into a single structure is illustrated on figure 248. In this installation the outlet works intake encircles the drop inlet tower of the spillway, and the outlet conduit extends along the top of the spillway conduit and empties into the latter some distance downstream from the spillway inlet.

The topography and geology of a site may have a great influence on the layout selection. Some sites may be suited only for a cut-and-cover conduit type of outlet works, while at other sites either a cut-and-cover conduit or a tunnel can be selected. Unfavorable foundation geology, such as deep overburdens or inferior foundation rock, will obviate the selection of a tunnel scheme. On the other hand, sites in narrow canyons with

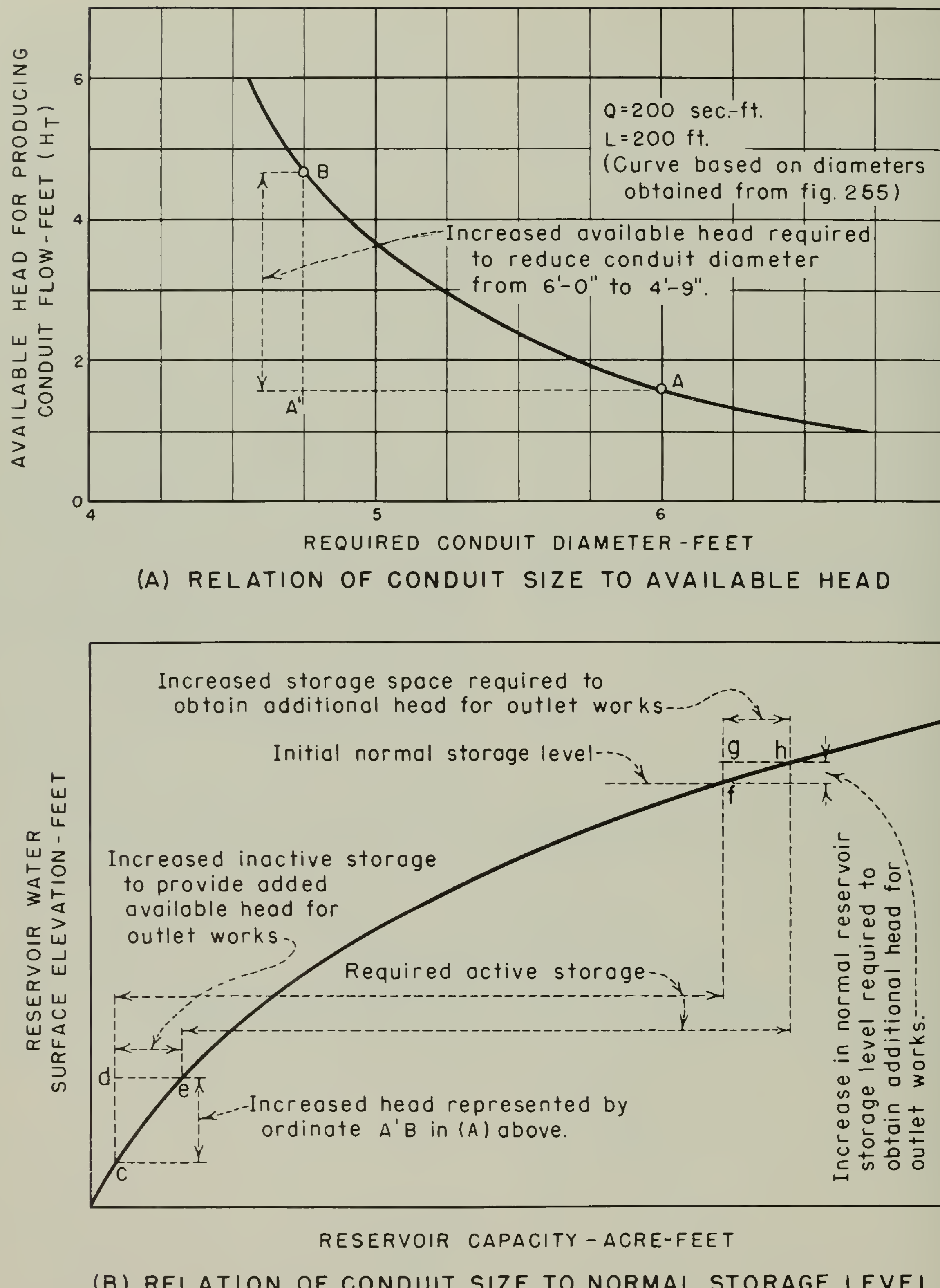


Figure 247. Relation of minimum design head to conduit size.

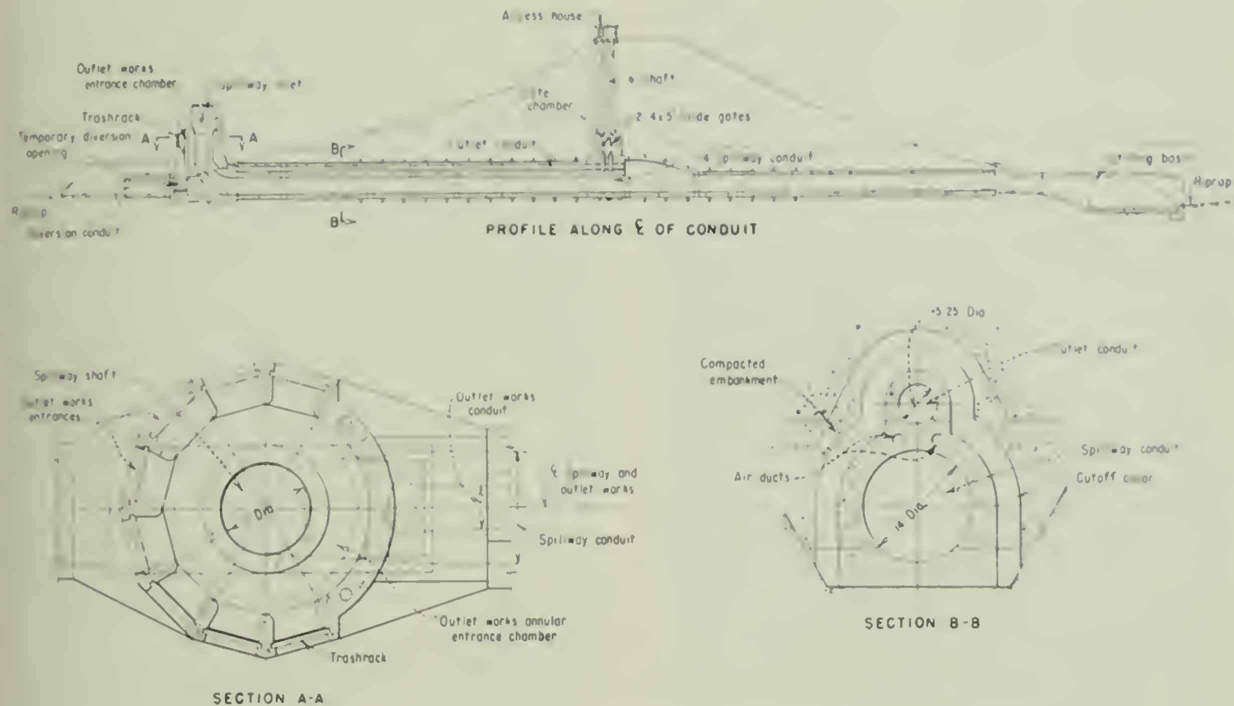


Figure 248. **Combined spillway and outlet works for Heart Butte Dam in North Dakota.**

steep abutments may make a tunnel outlet the only choice. Because of confined working space and excessive costs where hand construction methods must be employed, it is not practicable to make a tunnel smaller than about 6 feet in diameter. If constructed of precast material or if cast-in-place with the inside bore formed by a prefabricated liner, a cut-and-cover conduit can be constructed to almost any size. Thus the minimum size dictated by constructed conditions, as compared to the size established by hydraulic requirements, will have considerable influence on the choice of alternative cut-and-cover conduit or tunnel schemes.

Some sites favorable for a tunnel outlet may have unfavorable portal conditions which make it difficult to fit the inlet and exit structures to the remainder of the outlet works. In this situation, a central tunnel with cut-and-cover conduits leading to and away from the tunneled portion of the outlet may prove to be feasible. Such an arrangement is shown on figure 244 for Glen Anne Dam.

If water is to be taken from a reservoir for domestic use, special consideration must be given to the positioning of the intake. To assure the proper quality of the water, it may be necessary to draw from different levels of the reservoir at different seasons of the year or to restrict the draft to specific levels, depending on the reservoir stage. To prevent silt from being carried into the outlet system, intake locations at low points or pockets in the reservoir must be avoided. Similarly, intakes must not be placed at points in the reservoir where stagnant water or algae can accumulate or where prevailing winds will drift debris or undesirable trash to the intake entrance.

217. ***Arrangement of Outlet Works.*** An outlet works for a low dam, whether it is to divert water into a canal or release it to the river, often may consist of an open channel or a cut-and-cover structure placed at the dam abutment. The structure might consist of a conventional open flume or rectangular channel with a gate similar to that used for ordinary spillway installations, or it might be regulated by a submerged-type gate placed to close off openings in a curtain or head wall. Where the outlet is to be placed through a low earthfill embankment, a closed-type structure might be used which may consist of single or multiple units

of buried pipe or box culverts placed through or under the embankment. Flow for such an installation could be controlled by gates placed at the inlet or placed at an intermediate point along the conduit, such as at the crest of the embankment, where a shaft would be provided for gate operation. Downstream from the control structure, the channel would continue to the canal or to the river where, depending on the exit velocities which might prevail for the particular installation, a stilling device similar to one described in chapter VIII might be employed. Figure 240 shows typical installations of arrangements as described above.

For higher earthfill dams where an open channel outlet structure would not prove feasible, the outlet might be carried through, under, or around the dam as a cut-and-cover conduit or through the abutment as a tunnel. Depending on the position of the control device, the conduit or tunnel could be free flowing, flowing under pressure for a portion of its length, or flowing under pressure for its entire length. Intakes might be arranged to draw water from the bottom of the reservoir, or the inlet sills might be placed at some higher reservoir level. Dissipating devices similar to those described in chapter VIII could be utilized at the downstream end of the conduit. The outlet works also may discharge into the spillway stilling basin. Depending on the method of control and the flow conditions in the structure, access to the operating gates might be by bridge to an upstream intake tower, by shaft from the crest level of the dam, by walkway within the conduit or tunnel with entrance from the downstream end, or by a separate conduit or tunnel access adit. Typical arrangements as described above are illustrated on figures 241 through 244.

For a concrete dam the outlet works installation is usually carried through the dam as a formed conduit or a sluice, or as a pipe embedded in the concrete mass. Intakes and terminal devices can be attached to the upstream and downstream faces of the dam. Often the outlet is formed through the spillway overflow section, using a common stilling basin to dissipate both spillway and outlet works flows. Where an outlet works conduit is installed in the nonoverflow section of the dam or where an outlet must empty into a canal, a separate dissipating device will, of course, be necessary. Instead of a large single conduit, multiple smaller conduits might be utilized in a concrete dam to provide a less expensive as well as a more feasible arrangement for handling the outlet works releases. The conduits might be placed at a single level, or for added flexibility they may be positioned at several levels. Such an arrangement would reduce the cost of the control gates, because of the lower heads on the upper level gates. Typical outlet works installations for concrete dams are shown on figure 245.

Where a diversion tunnel is utilized during the construction of a concrete dam, it is often feasible to convert the tunnel into a permanent outlet works by providing outlet sluices or conduits through the tunnel plug. Ordinarily, the diversion tunnel for a concrete dam will be in good quality rock and will therefore require a minimum of lining protection. Further, the outlet portal of the tunnel will generally be located far enough downstream from the dam so that no dissipating structure will be needed, or at most only a deflector will be required to direct the flow to the downstream river channel.

218. *Location of Outlet Works Controls.*—(a) *General.*—Where an outlet works is ungated, as will be the case with many detention dams, the conduit will act similarly to a culvert spillway, as described in section 206. Where water must be stored and the release regulated at specific rates, control gates or valves will need to be installed at some point along the conduit.

Gates and valves for outlet works are categorized according to their functional use in the structure. Operating gates and regulating valves are used to control and regulate the outlet works flow and are designed to operate in any position from closed to fully open. Guard or emergency gates are designed to be utilized only to effect closure in the event of failure of the operating gates, or where unwatering is desired either to inspect the conduit below the guard gates or to inspect and repair the operating gates. Occasionally slots are provided at the conduit entrance to accommodate stoplogs or bulkheads so that the conduit can be closed off during an emergency period. For such installations, guard gates may or may not be provided, depending on whether or not the stoplogs can be placed readily if an emergency arises during normal reservoir operating periods.

The control gate for an outlet works can be placed at the upstream end of the conduit, at an

intermediate point along its length, or in some instances at the lower end of the structure. Where flow from a control gate is released directly into the open as free discharge, only that portion of the conduit upstream from the gate will be under pressure. Where a control gate or valve discharges into a closed pressure pipe, the control will serve only to regulate the releases; full pipe flow will occur in the conduit both upstream and downstream from the control gate. For the pressure-pipe type, the location of the gate or valve will have little influence on the design insofar as internal pressures are concerned. However, where a control discharges into a free-flowing conduit, the location of the control gate becomes an important consideration in the design of the outlet. The effects of locating the control at various positions in a conduit are discussed in the following subsections.

(b) *Control at Upstream End of Conduit.*—For an outlet works with an upstream control discharging into a free-flow conduit, part full flow will occur throughout the length of the structure. Ordinarily, the operating head and the conduit slope will result in flow at supercritical stage. The structural design of the conduit and the safety and practical aspects of the layout will then be concerned only with the effects of external loadings and outside water pressures on the structure. Along the upstream portion of the conduit and extending until sufficient rock cover is available over a tunnel or until an adequate thickness of impervious embankment is obtained over a cut-and-cover conduit, practically full reservoir head will be exerted against the outside of the conduit barrel. The conduit walls must therefore be designed to withstand such pressures, and details of design must be selected to preserve the watertightness of the conduit. For a cut-and-cover conduit where settlement of the structure (due to foundation consolidation with increasing embankment load) must be anticipated, special care must be taken in design details to prevent the cracking of the conduit barrel and to seal any formed joints, since cracks and open joints will invite excessive leakage or piping of surrounding embankment material into the conduit.

With the controls placed at the upstream end of a conduit, fishscreens, stoplog slots, trashracks, guard gates, and regulating gates or valves can all be combined in a single intake structure. This arrangement will simplify outlet works operation by centralizing all control features at one point. Further, the entire conduit may be readily unwatered for inspection or repair. The intake will consist of a tower rising from the base of the outlet conduit to an operating deck placed above maximum reservoir water level, with the tower located in the reservoir area near the upstream toe of the dam. Access to the structure operating deck will then be possible only by boat, unless an access bridge is provided from the reservoir shore or from the crest of the dam. The intakes at Crane Prairie and Crescent Lake Dams (fig. 241) illustrate typical tower arrangements. Figure 19 is a photograph of the intake tower and access bridge at Crescent Lake Dam. Figure 92 shows the intake tower at Crane Prairie Dam.

(c) *Control at Intermediate Point Along Conduit.*—Where a control gate is placed at an intermediate point along a conduit and discharges freely into the downstream section or where the flow is conveyed in a separate downstream pipe, the internal pressure upstream from the control will be approximately equal to full reservoir head. The structural design and safety aspects of the upstream portion will then be concerned with the effects of both the external loadings and the internal hydrostatic pressure acting on the conduit shell. The watertightness of the conduit in the extreme upstream section will be of less importance because the external and internal hydrostatic pressures will closely balance, and leakage into or out of the conduit will be minimized. However, the external pressure around the conduit will normally diminish with increasing distances from the reservoir. At downstream portions of the pressure conduit, there may be an excess of internal pressure which could cause leakage through joints or cracks into the material surrounding the conduit barrel. The flow from such leaks might follow along the outside of the conduit to the section not under pressure where piping through joints could occur. Where a pressure conduit is carried through an embankment, the development of piping with eventual failure of the dam is a possibility. Where such a conduit comprises a tunnel, leakage through

seams in the rock might saturate the hillside overburden above the tunnel and cause a sloughing or landslide on the abutment.

To minimize the possibilities of failures such as those described above, it is normal practice to limit the length of the pressure portion of a cut-and-cover conduit to that part of the outlet upstream from the crest of the dam, or, in some instances, to approximately the upstream one-third of the dam only. Where there is concern regarding the watertightness of a pressure conduit in the upstream portion of a dam, but there are compelling reasons why the control cannot be located near the upstream end of the conduit, that portion upstream from the control may be provided with a steel liner. Such a detail was employed at Shadehill Dam (fig. 243).

For a tunnel installation, except for the possibilities of leakage discussed previously, the location of the control gate is not as critical as it is for cut-and-cover outlets. However, the pressure portion of the tunnel ordinarily should not extend downstream beyond a point where the weight of the column of rock above the tunnel or the side resistance to a blowout is less than the internal pressure forces, unless the tunnel lining is properly reinforced to withstand the internal pressure and a waterproof liner is provided to prevent a buildup of hydrostatic pressures outside the lining.

There may be cases where neither pressure nor free flow is desirable, either for a portion of a conduit or for its entire length. Such instances may occur where it is expected that excessive settlement or movement of the conduit will occur and that cracking and opening of joints cannot be avoided. In this situation, to forestall serious leakage that would occur if a free-flow or pressure conduit were used, a separate small steel pipe can be installed inside the larger conduit to convey the flow. The control gate or valve could be installed at the upper end of the pipe, at some intermediate location, or at the downstream end If the control gates are not placed at the upstream end, guard gates might be provided at the upstream end of the pipe to effect closure in the event of a leak or failure along any part of the pipe.

Where a control gate discharges into a free-flow conduit, an access and operating shaft extending from the conduit to a level above high water surface in the reservoir will be required. For a cut-and-cover conduit under an earthfill dam, the location of the control gates is usually selected so that the operating shaft is positioned immediately upstream from the crest of the dam (fig. 243). Where flows in the downstream portion of the conduit are carried in a separate pipe, a control chamber is usually provided at the upstream end of the pipe (fig. 242).

The control gates or valves for a conduit or sluice through a concrete dam can be positioned at any point, either upstream to afford free flow in the sluice or at the downstream end to provide pressure pipe flow. Where the sluices are placed in the overflow section of the dam, upstream gates controlling the entrance or valves operated from an interior gallery in the dam are ordinarily employed. Where the outlets are placed in the nonoverflow section, either upstream gates or downstream valves are utilized (fig. 245).

B. OUTLET WORKS COMPONENTS

219. ***General.***—For an open channel outlet works or for a conduit-type outlet where part full flow prevails, the control gates or valves are the determining factors which establish the outlet works capacity. Where an outlet works operates as a pressure pipe, the size of the waterway as well as that of the control device determines the capacity.

The overall size of an outlet works is determined by its hydraulic head and the required discharge capacity. The selection of the size of some of the component parts of the structure, such as the tunnel, is dictated by practical considerations or by collateral requirements such as diversion. Since, as discussed in part C of this chapter, the capacity of a closed system outlet is influenced by the hydraulic losses through the components, the sizes of various features can be changed in relation to one another for a given capacity. For example, a streamlined inlet may permit the installation of a smaller gate for a given size conduit, or a larger gate may allow the use of a

smaller conduit. Or, for a given discharge, enlargement of the upstream pressure conduit of a closed pipe system may permit reduction in the size of the downstream pressure pipe and consequently in the size of the downstream conduit. The determination of the best overall layout to achieve economy in the design may, therefore, require alternative studies involving various trial sizes of the different components of the outlet works.

When the type of waterway is chosen and the method of control is established, the associated structures to complete the layout can be selected. The type of intake structure will depend on its location and function and on the various appurtenances such as fishscreens, trashracks, stoplog arrangements, or operating platforms which must be furnished. A means for dissipating the energy of flow before returning the discharge to the river may have to be provided. This might be accomplished by a deflector lip, a stilling basin, or a similar dissipator device. Gate chambers, control platforms, or enclosures may be required to provide operating space and protective housing for the control devices. An outlet works also may require an entrance channel to lead diversion flows or flows when the reservoir is low to the intake structure, and an outlet channel to return releases to the river.

220. *Waterways.*— (a) *Open Channels.*—Open channel waterways for outlet works are similar to those for a canal headworks structure, a sluiceway through a dam, or an ordinary spillway. The waterway will usually consist of a channel or flume placed through the embankment to carry the flow from the reservoir to a canal or to the downstream river level. Details of the design are comparable to those for a gated orifice-controlled spillway.

(b) *Tunnels.*—Because of its inherent advantages, a tunnel outlet works is preferred where abutment and foundation conditions will permit its utilization and if it is economical compared with other types. A tunnel is not in direct contact with the dam embankment, and therefore it provides a much safer and more durable layout than can be achieved with either a cut-and-cover conduit or an open channel structure. A minimum of foundation settlement, differential movement, and structural displacement will be experienced with a tunnel which has been bored through competent abutment material, and seepage along the outer surfaces of the tunnel lining or leakage into the material surrounding the tunnel will be less serious. Furthermore, there is less likelihood that failure of some portion of the tunnel would cause failure of the dam than if a cut-and-cover conduit passed under or through the dam.

484556 O—60 —25

Ordinarily, pressure tunnels in competent rock do not require lining reinforced to withstand full internal hydrostatic pressures, since the surrounding rock normally can assume such stresses. If the rock cover has sufficient weight and enough side resistance to prevent blowouts, only an unreinforced lining is necessary to provide watertightness in seamy rock and smoother surfaces for better hydraulic flow.

Where pressure tunnels are placed through less competent foundations, such as jointed or yielding rock, the tunnel lining must be designed to withstand external rock loadings in addition to internal hydrostatic pressures. At the extreme upstream end of an outlet works tunnel, where external hydrostatic pressures may nearly balance the internal pressures, the lining will need to be reinforced to withstand rock loads only. At the downstream portions of the tunnel, where outside water pressures diminish, the design of the tunnel lining will need to consider both external loads from the rock and internal water pressures.

For free-flow tunnels in competent rock, lining might be provided only along the sides and bottom to form a smooth waterway. In less competent material, lining of the complete cross section may be necessary to prevent caving. For that portion of a free-flow tunnel immediately adjacent to the reservoir or just downstream from a pressure tunnel, cognizance must be taken of the possibility of hydrostatic pressure buildup behind the lining due to leakage through the walls of the pressure tunnel or to seepage from the reservoir. Ordinarily, such external water pressure can be reduced by grouting and by providing drain holes through the lining of the free-flow tunnel.

The need for lining a tunnel in which an independent pipe is installed depends entirely on the competency of the rock to stand unsupported. Since such a tunnel is used to house the pressure pipe and provide access to an upstream gate, lining sufficient to avoid rock falls might be provided for protection of the pipe and operating personnel.

For a pressure tunnel a circular cross-sectional shape is the most efficient, both hydraulically and structurally. For a free-flow tunnel a horseshoe shape or a flat bottom tunnel will provide better hydraulic flow, but it is not as efficient as the circular shape for carrying external loads. For small tunnels under only moderate heads the horseshoe-shaped pressure tunnel and either the horseshoe or the flat-bottomed free-flow tunnel may be permissible, depending on foundation conditions. As discussed in section 216, it is not practicable to provide tunnels much smaller than about 6 feet in diameter. The structural design of tunnels, including reinforcement of linings, is discussed in section 234.

(c) *Cut-and-Cover Conduits.*—If a closed conduit is to be provided and if foundation conditions are not suitable for a tunnel, or if the required size of the waterway is too small to justify the minimum-size tunnel, a cut-and-cover conduit must be used. Since such a conduit passes through or under the dam, conservative and safe designs must be used. Numerous failures of earthfill dams caused by improperly designed or constructed cut-and-cover outlet conduits have demonstrated the need for conservative procedures.

A conduit should be placed on the most competent portion of the dam foundation. Details of the design must allow for expected settlement, shrinkage, and lateral or longitudinal displacement without interfering with the continuity of the structure which must provide a safe and leakproof waterway.

Where bedrock occurs at the site, every attempt should be made to place the entire conduit on such a foundation. If this is not physically or economically feasible, the structure should be located where overburden is shallow so there will be a minimum of foundation settlement. If a uniform foundation exists and it is determined that foundation settlement will not be excessive, the excavation for the conduit should be to exact grade and the conduit supported on undisturbed material. Where the conduit foundation in its natural state is not suitable, the unsuitable material should be excavated to a depth where a material competent to support the load is reached, and the excavation should be refilled with compacted material of desired stability and impermeability. Unsuitable foundation materials include those which are so permeable as to permit excessive seepage, those subject to excessive settlement on loading, and those subject to settlement on saturation of the foundation by the reservoir. These materials are described in chapter V. In all cases, regardless of the nature of the foundation, the contact of the conduit with the foundation must provide a watertight bond, free of void spaces or unconsolidated areas.

Cut-and-cover conduits must be designed with sufficient strength to withstand the load of the fill overlying the structure. Pressure conduits must also be designed to resist an internal hydrostatic pressure loading equal to full reservoir head. Design loadings for conduits are further discussed in section 235.

The adaptability of a cut-and-cover conduit and the desirability of utilizing such a conduit as a pressure pipe or as a free-flow waterway are discussed in section 218. Since a cut-and-cover conduit in most instances must be constructed before the embankment, conduit settlement will follow the foundation settlement resulting from the embankment loading. The conduit settlement therefore will be maximum at the point of highest fill and will diminish toward each end. Structure details must be selected to allow for such settlement, and conduit profiles must be adjusted to take account of the drop in grade near the center of the dam. Joint treatment and reinforcement requirements are discussed in section 235.

221. *Controls.*—(a) *Control Devices.*—Selection of the outlet works arrangement for small dams should be based on the use of commercially available gates and valves or relatively simple gate designs, rather than on the use of special devices which will involve expensive design and fabrication costs. Cast-iron slide gates, which may be used for control and guard gates, are available for both rectangular and circular openings and for design heads up to 50 feet. Simple radial gates are available for ordinary surface installations, and top-seal radial gates can be secured from manufacturers on the basis of simple designs and specifications. For low heads, commercial gate valves and butterfly valves are suitable for control at the downstream end of pressure pipes if they are designed to operate under free discharging conditions. They are also suitable as inline guard valves for wide-open operation, and they can be adopted for inline control valves if air venting

of the pipe is provided immediately downstream from the valve.

(b) *Arrangement of Controls.*—Flows through low-head outlet works can be controlled by various devices, as shown on figure 240. A surface radial gate may be installed in an open channel, as shown for Putah Diversion Dam. Top-seal radial gates installed at the entrance or within a culvert outlet works are shown for Flatiron Dam and for Camp Creek and Bartley Diversion Dams. Slide gates, similar to those shown for Woodston and Fort Sumner Diversion Dams, may be used to control flows through either open channel or culvert outlet works which are provided with headwall structures.

Upstream gate controls for conduits are generally placed in a tower structure, with the gate hoists mounted on the operating deck (fig. 241). With this arrangement the tower must extend above the maximum water surface.

If controls are located at some intermediate point along a conduit, slide gates or top-seal radial gates can be used, operating in a wet well shaft which extends vertically from the conduit level to the level of the crest of the dam. These arrangements are typified by the installations shown on figure 243.

A variation of the slide gate control which will eliminate the need for a wet well shaft is possible. In this instance watertight bonnets are provided over the gate slots and the gates are operated either from a dry shaft or from an operating chamber located above the conduit level. Watertight bushings are provided where the gate stems extend through the bonnets.

Valves also can be used as controls at intermediate points along conduits. A dry well is provided and the valve is placed in a length of pipe whose upstream end is encased in a concrete plug. This type of installation is illustrated in figure 242. If the flow is carried by separate pipe in a conduit sufficiently large to afford access along the pipe from the downstream end, a domed chamber can be used rather than a dry well shaft. Such a chamber is provided at Soldier Canyon Dam as shown on figure 244.

If a concrete dam utilizes a slide gate control on its upstream face, the gate frame and stem guides can be mounted directly on the concrete face and the hoist can be placed on a platform cantilevered from the crest of the dam. If the gate is placed at an intermediate point along a conduit formed through the concrete dam, the gate can be operated either in a wet well with the hoist placed at the crest of the dam, or from a gallery if the watertight bonnet cover arrangement is provided over the gate well. Inline valves can also be operated from the gallery or from a chamber formed inside the dam. A control valve placed on the end of the conduit at the downstream face of the dam can be operated from a platform extending from the face of the dam. Typical installations are illustrated on figure 245.

(c) *Control and Access Shafts.*—Where a free-flow conduit is provided downstream from the control devices, access for operating is usually from a shaft located directly over the controls. If the wet well arrangement is utilized, a shaft of sufficient width and breadth to accommodate the several wells must be provided. When the type of controls permits dry well installations, only sufficient space to provide operating room at the bottom of the shaft is needed. A smaller access shaft, either directly above or offset from the chamber, and just large enough to permit passage of removable and replaceable gate parts, will then be needed.

The operating or access shaft for a tunnel outlet works can be sunk into the undisturbed hillside and lined with concrete as necessary to maintain the shaft walls intact. Where such a shaft is used for access and ventilation only, a minimum of wall lining will be needed. Where an access shaft is to be used for a wet well arrangement, adequate lining to make the shaft reasonably watertight will be required. If a cut-and-cover conduit scheme is used, the shaft must be constructed through the dam embankment. The structural design must consider the possibility of settlement and lateral displacement as a result of movement of the embankment. Where a wet well shaft is employed, care must be taken in the design to prevent cracks and the opening of joints which would permit leakage from the interior of the shaft into the surrounding embankment. The walls of the wet well shaft must be designed to resist internal hydrostatic pressure from full reservoir head in addition to the external embankment loading. If a shaft extends through the embankment and projects into the reservoir, external hydrostatic loads must also be considered. The protruding portion of the shaft constitutes a tower

which is subject to the ice loads discussed in section 222.

(d) *Control Houses.*—A housing around the outlet controls is sometimes provided where operating equipment would otherwise be exposed or where adverse weather conditions will prevail during operating periods. A house is sometimes provided to enclose the top of an access shaft, although the controls may be located elsewhere. Such houses are usually made sufficiently large to accommodate auxiliary equipment such as ventilating fans, heaters, flow measuring and recording meters, air pumps, small power-generator sets, and equipment needed for maintenance.

222. *Intake Structures.*—In addition to forming the entrance into the outlet works, an intake structure may accommodate control devices. It also supports necessary auxiliary appurtenances (such as trashracks, fish screens, and bypass devices), and it may include temporary diversion openings and provisions for installation of bulkhead or stoplog closure devices.

An intake structure may take on many forms, depending on the functions it must serve as noted above, on the range in reservoir head under which it must operate, on the discharge it must handle, on the frequency of reservoir drawdown, on the trash conditions in the reservoir which will determine the need for or the frequency of cleaning of the trashracks, on reservoir ice conditions or wave action which could affect the stability, and on other such considerations. An intake structure may either be submerged or extended as a tower to some height above the maximum reservoir water surface, depending on its function. A tower must be provided if the controls are placed at the intake, or if an operating platform is needed for trash raking, maintaining and cleaning of fish screens, or installing stoplogs. Where the structure serves only as an entrance to the outlet conduit and where trash cleaning ordinarily will not be required, a submerged structure can be adopted.

The conduit entrance can be placed vertically, inclined, or horizontally, depending on intake requirements. Where a sill level higher than the conduit level is desired, the intake can be a drop inlet similar to the entrance of a drop inlet spillway. A vertical entrance is usually provided for inlets at the conduit level. In certain instances at small installations where the gate is placed and operated on the upstream slope of a low dam, an inclined entrance can be adopted. Such an arrangement is typified by the Ortega Reservoir outlet shown on figure 246. In most cases conduit entrances should be rounded or bellmouthed to reduce hydraulic entrance losses.

The necessity for trashracks on an outlet works depends on the size of the sluice or conduit, the type of control device used, the nature of the trash burden in the reservoir, the utilization of the water, the need for excluding small trash from the outflow, and other factors. These factors will determine the type of trashracks and the size of the openings. Where an outlet consists of a small conduit with valve controls, closely spaced trashbars will be needed to exclude small trash. Where an outlet involves a large conduit with large slide gate controls, the racks can be more widely spaced. If there is no danger of clogging or damage from small trash, a trashrack may consist simply of struts and beams placed to exclude only the larger trees and such floating debris. The rack arrangement will also depend on accessibility for removing accumulated trash. Thus, a submerged rack which seldom will be unwatered must be more substantial than one which is at or near the surface. Similarly, an outlet with controls at the entrance where the gates can be jammed by trash protruding through the rack bars must have a more substantial rack arrangement than if the controls are not at the entrance.

Trash bars usually consist of thin, flat steel bars which are placed on edge from 3 to 6 inches apart and assembled in rack sections. The required area of the trashrack is fixed by a limiting velocity through the rack, which in turn depends on the nature of the trash which must be excluded. Where the trashracks are inaccessible for cleaning, the velocity through the racks ordinarily should not exceed 2 feet per second. A velocity of up to approximately 5 feet per second may be tolerated for racks which are accessible for cleaning.

Trashrack structures also may take on varied shapes, depending on how they are mounted or arranged on the intake structure. Trashracks for a drop inlet intake are generally formed as a cage surmounting the entrance. They may be arranged as an open box placed in front of a vertical entrance or they may be positioned along the front side of a tower structure. Figures 240

through 246 illustrate various arrangements of trashracks at entrances to outlet works.

At some reservoir sites it may be desirable or required to screen the inlet entrance to prevent fish from being carried through the outlet works. Several such installations are illustrated on figure 241. Because small openings must be used to exclude fish, the screens can easily become clogged with debris. Provisions must therefore be made for periodically removing the screens and cleaning them by brooming or water jetting.

Where the control is placed at an intermediate point along a conduit, some means of unwatering the upstream pressure section of the conduit and the intake is desirable to make inspections and needed repairs. Stoplog or bulkhead slots are generally provided for this purpose in the intake or immediately downstream from the intake. In intake towers containing control devices, the stoplog slots are placed upstream from the controls. A circular, flat bulkhead which can drop down over the entrance is generally provided for a drop inlet structure. The bulkhead can be stored on supports near the top of the structure. Closure can then be effected under water by lowering the bulkhead with a cable winch operated from a barge, or from the top of the structure if the reservoir is low enough to expose the upper portion.

For an intake structure with inlet sill above the invert of the conduit, it may be desirable for various reasons to draw the reservoir down below that level. In such an instance a bypass can be provided near the base of the structure to connect the reservoir to the conduit downstream. In other instances, where flow must be maintained while installing or maintaining the control gates and outlet pipes or while repairing or maintaining the free-flow conduit concrete, it may be desirable to carry a separate pipe under or alongside the conduit to bypass it entirely. In either case, the bypass inlet can be placed in the intake structure and usually can be controlled by a slide gate mounted on one of the faces of the structure and operated from some higher level.

Where winter reservoir storage is maintained and the surface ices over, the effect of such conditions on the intake structure must be considered. Where reservoir surface ice can freeze around an intake structure, there is danger to the structure not only from the ice pressures acting laterally but also from the uplift forces if a filling reservoir lifts the ice mass vertically. These effects must be considered when the advantages or disadvantages of a tower are compared with those of a submerged intake. Where a tower design is adopted and ice conditions present a hazard, an air bubbling system can be installed around the base of the structure to circulate the warmer water from the bottom of the reservoir which will keep the surface area adjacent to the structure free of ice. Such a system will require a constant supply of compressed air and must be operated continuously during the winter months.

223. *Terminal Structures and Dissipating Devices.* The discharge from an outlet, whether through gates, valves, or free-flow conduits, will emerge at a high velocity, usually in a nearly horizontal direction. For a free-flow conduit, deflector devices might be employed to direct the high-velocity flow away from the outlet structure and past the downstream toe of the dam if erosion-resistant bedrock exists at shallow depths in the downstream channel. Where softer foundations exist, a dissipating device might be provided to absorb the energy of flow before it is returned to the river or canal. The flow from valves at the end of an outlet will generally be in the form of a jet, which can be discharged directly into the river, into a plunge basin downstream from the outlets, or into a hydraulic jump-type basin.

Where an outlet is terminated as a submerged pipe, a stilling well dissipator is sometimes employed to dissipate the flow energy. This device consists of a vertical water-filled well in which dissipation is achieved by turbulence and by diffusion of the incoming flow. The incoming flow can be directed horizontally into the well near the bottom, or it may be directed vertically downward into the well through a vertical pipe and released near the bottom. In both cases the flow rises upward and emerges out of the top of the well.

Terminal structures for free-flow conduit outlet works are essentially the same as those for spillways. The hydraulic designs of such basins are discussed in part E of chapter VIII. The design of basins to dissipate jet flow and the design of stilling wells to dissipate submerged pipe flow are discussed in section 230.

224. *Entrance and Outlet Channels.* An entrance channel leading to the outlet works intake

and an outlet channel to deliver flow to the river downstream, are often required with a tunnel or cut-and-cover conduit layout. An entrance channel may be required to convey diversion flows to a conduit placed in an abutment, or to deliver water to the outlet works intake during low reservoir stage. Outlet channels may be required to convey discharges from the end of the outlet works to the river downstream or to a canal. All such channels should be excavated to stable slopes and to dimensions which will provide nonscouring velocities. Entrance channel velocities are usually made less than those through the thrashracks, and the entrance channel is often widened near the intake structure to permit a smooth, uniform flow into all trashrack openings.

The outlet channel dimensions and the need for lining or riprap protection will depend on the nature of the material through which the channel is excavated. Occasionally a control or a measuring station is placed in the outlet channel, in which event the selection of the grade and cross section of the channel becomes an important consideration. The effects of aggradation or degradation of the main river channel must be considered in selecting the outlet works outlet channel dimensions.

C. HYDRAULIC DESIGN

225. *Nature of Flow in Outlet Works.*—The hydraulics of outlet works usually involve either one or both of two conditions of flow—open channel (or free) flow and full conduit (or pressure) flow. Analysis of open channel flow in outlet works, either in an open waterway or in a part full conduit, is based on the principle of steady nonuniform flow conforming to the law of conservation of energy. Full pipe flow in closed conduits is based on pressure flow, which involves a study of hydraulic losses to determine the total heads needed to produce the required discharges.

Hydraulic jump basins, baffle or impact block dissipators, or other stilling devices normally are employed to dissipate the energy of flow at the downstream end of the outlet works. Many of these devices are designed on the basis of the law of conservation of momentum.

226. *Open Channel Flow in Outlet Works.*—Flow in an open channel outlet works will be similar to that in open channel spillways, which is discussed in chapter VIII. Where unsubmerged radial or slide gates are used, discharges through the control with the gates completely raised will be open crest flow as computed by equation (3) of chapter VIII:

$$Q=CLH^{3/2}$$

Discharge coefficients applicable to various crest arrangements are discussed in section 190.

When open channel outlet flow is controlled by partly open surface gates, or where top-seal radial gates or submerged slide gates control the flow sluice flow will result. Discharges for such flow are given by equation (7) of chapter VIII:

$$Q=\frac{2}{3}\sqrt{2g}\ CL\,(H_1^{3/2}-H_2^{3/2})$$

Discharge coefficients for sluice control can be determined from figure 197 or table 30 (sec 228(d)).

In instances where there is high tailwater due to canal water surfaces or to downstream influences in the streambed, the control openings may be partly or entirely submerged. For such conditions the discharge through the control will be in accordance with submerged orifice or tube flow as computed by the equation:

$$Q=CA\sqrt{2gH} \qquad (1)$$

where:

A=the area of the opening,

H=the difference between the upstream and downstream water levels, and

C=the coefficient of discharge for submerged orifice or tube flow.

Coefficients for various conditions of orifice suppression and tube geometry can be evaluated from figure 249, or from published data in various hydraulic handbooks [1, 2] [2] and textbooks.

Flow in an open channel downstream from the headworks will be at either subcritical or super-

[2] Numbers in brackets refer to items in the bibliography, sec. 237.

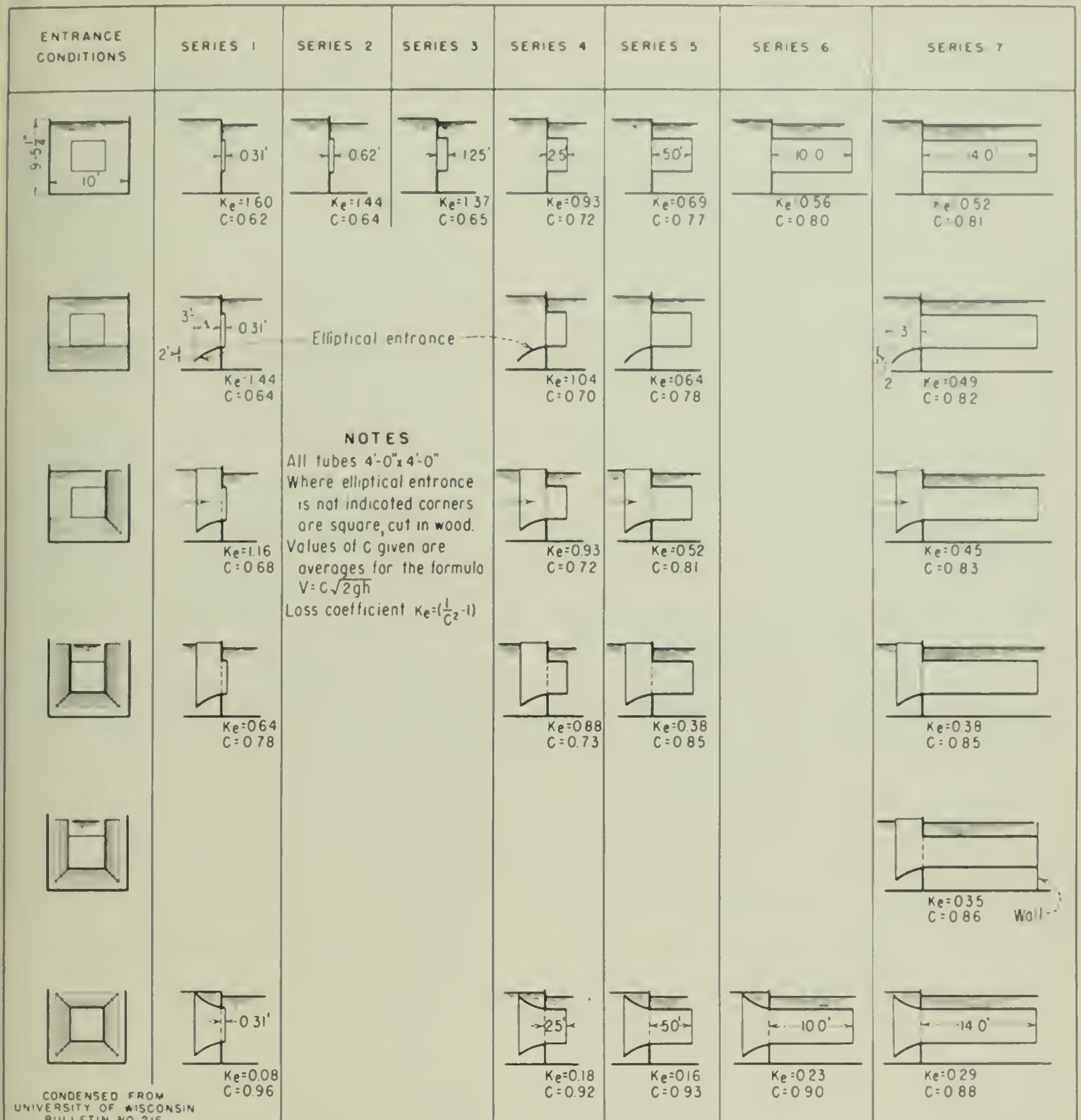


Figure 249. **Flow through submerged tubes.**

critical stage, depending on the flow conditions through the control structure. In either case, flow depths and velocities throughout the channel can be determined from Bernoulli's equation, as discussed in section 196.

Flow in ungated outlet conduits will be similar to that in a culvert spillway, as discussed in section 206. Where the inlet geometry and the conduit slope are such that the control will remain at the inlet, part full flow will prevail and flow depths and velocities will be in accordance with the Bernoulli theorem for open channel flow. When flow from a pressure conduit discharges into a free-flow conduit, the flow in the latter most often will be at supercritical stage, with flow depths and velocities comparable to those which would prevail in an open channel. Computation procedures to determine the flow conditions

according to the Bernoulli equation are presented in section 196.

Outlet conduits flowing partly full should be analysed using maximum and minimum assumed values of the coefficient of roughness, n, when evaluating the required conduit size and the energy content of the flow as is done for spillway design (see sec. 196). To be assured of a sufficient conduit size to allow for air swell and surges, values of n of about 0.018 should be assumed in computing the depth or area of flow in a concrete-lined conduit. For computing the energy of flow at the end of the conduit to determine dissipator design, a value of n of about 0.008 should be assumed. To assure a free surface in the conduit for all stages of flow, and to guarantee against sealing of some portion from splashing or surging, the conduit should be designed to flow not more than 75 percent full at maximum capacity.

Terminal deflectors or energy dissipating devices placed at the downstream end of free-flow outlet conduits will be similar to those discussed in part E of chapter VIII for spillways. Transitions to diverge the flow from the conduit portal to the stilling device and the allowable convex curvature of the floor entering the stilling device are determined as discussed in section 197.

227. *Pressure Flow in Outlet Conduits.*—If a control gate is placed at some point downstream from the conduit entrance, that portion above the control gate will flow under pressure. An ungated conduit may also flow full depending on the inlet geometry. The phenomena and the hydraulic equations for flow through an ungated conduit under pressure are discussed in section 206. The hydraulic design of a gated pressure conduit is similar to that for an ungated pressure conduit, discussed in section 206.

For flow in a closed pipe system, as shown on figure 250, Bernoulli's equation can be written as follows:

$$H_T = h_L + h_{v_2} \tag{2}$$

where:

H_T= the total head needed to overcome the various head losses to produce discharge, and

h_L= the cumulative losses of the system.

Equation (2) can be expanded to list each loss, as follows:

$$H_T = h_t + h_e + h_{b_5} + h_{f_5} + h_{ex_{(5-4)}} + h_{f_4} + h_{c_{(4-3)}}$$
$$+ h_{g_3} + h_{ex_{(3-1)}} + h_{f_1} + h_{b_1} + h_{c_{(1-2)}} + h_{g_2} + h_{v_2} \tag{3}$$

where:

h_t= trashrack losses,
h_e= entrance losses,
h_b= bend losses,
h_c= contraction losses,
h_{ex}= expansion losses,
h_g= gate or valve losses,
h_f= friction losses, and
h_v= velocity head exit loss at the outlet.

In the above equation the number subscripts refer to the various components, transitions, and reaches to which head losses apply.

For a free-discharging outlet, H_T is measured from the reservoir water surface to the center of the outlet gate or the outlet opening. If the outflowing jet is supported on a downstream floor the head is measured to the top of the emerging jet at the point of greatest contraction; if the outlet portal is submerged the head is measured to the tailwater level.

Where the various losses are related to the individual components, equation (3) can be written:

$$H_T = K_t\left(\frac{v_6^2}{2g}\right) + K_e\left(\frac{v_5^2}{2g}\right) + K_{b_5}\left(\frac{v_5^2}{2g}\right) + \frac{fL_5}{D_5}\left(\frac{v_5^2}{2g}\right)$$
$$+ K_{ex}\left(\frac{v_5^2}{2g} - \frac{v_4^2}{2g}\right) + \frac{fL_4}{D_4}\left(\frac{v_4^2}{2g}\right) + K_c\left(\frac{v_3^2}{2g} - \frac{v_4^2}{2g}\right)$$
$$+ K_g\left(\frac{v_3^2}{2g}\right) + K_{ex}\left(\frac{v_3^2}{2g} - \frac{v_1^2}{2g}\right) + \frac{fL_1}{D_1}\left(\frac{v_1^2}{2g}\right) + K_{b_1}\left(\frac{v_1^2}{2g}\right)$$
$$+ K_c\left(\frac{v_2^2}{2g} - \frac{v_1^2}{2g}\right) + K_g\left(\frac{v_2^2}{2g}\right) + K_v\left(\frac{v_2^2}{2g}\right) \tag{4}$$

where:

K_t= trashrack loss coefficient,
K_e= entrance loss coefficient,
K_b= bend loss coefficient,
f= friction factor in the Darcy-Weisbach equation for pipe flow (discussed in sec. 228 (b)),

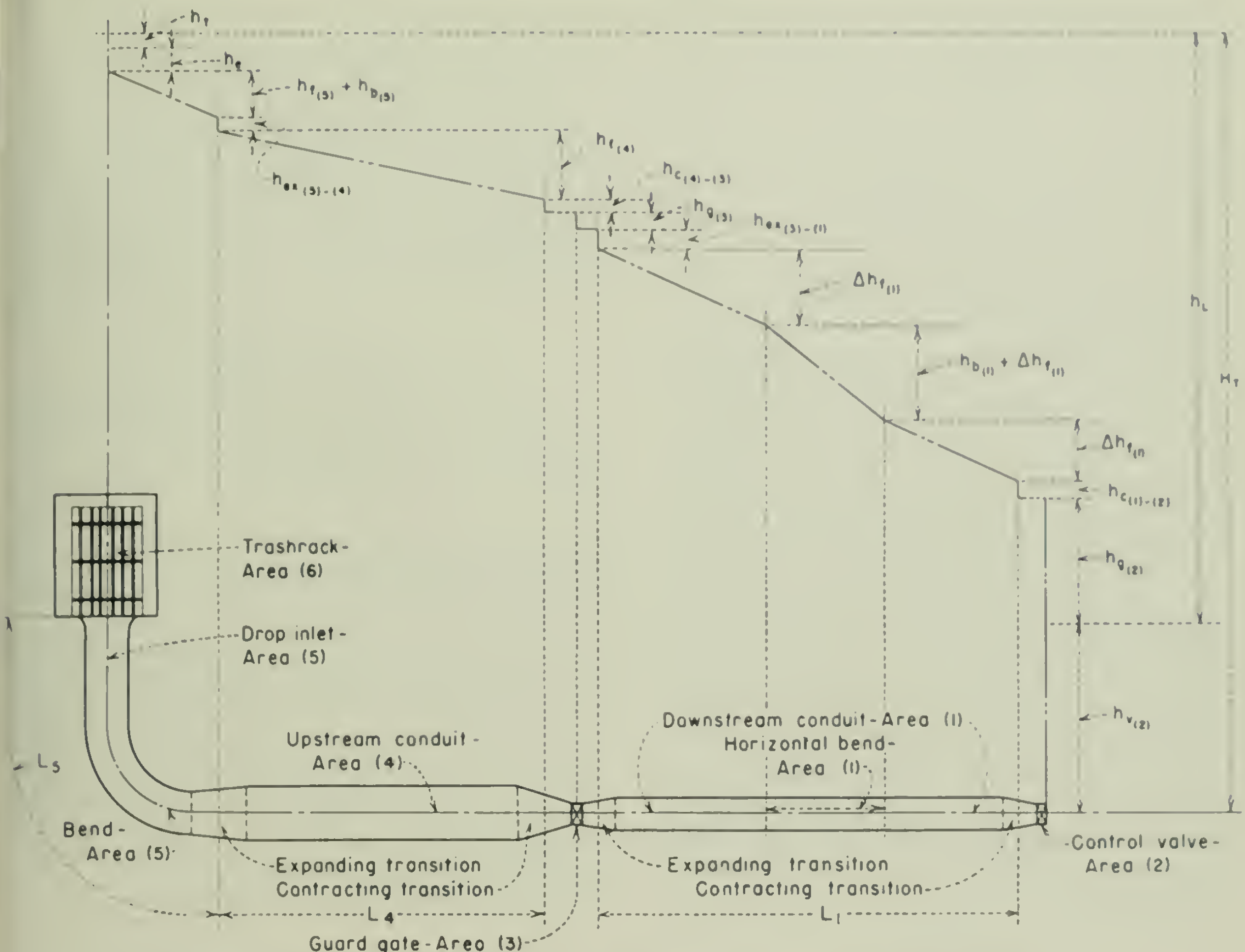


Figure 250. **Pictorial representation of head losses in conduit flowing under pressure.**

K_{ex} = expansion loss coefficient,
K_c = contraction loss coefficient,
K_g = gate loss coefficient, and
K_v = exit velocity head coefficient at the outlet.

Equation (4) can be simplified by expressing the individual losses in terms of an arbitrarily chosen velocity head. This velocity head is usually selected as that in a significant section of the system. If the various velocity heads for the system shown on figure 250 are related to that in the downstream conduit, area (1), the conversion for "x" area is found as follows:

Since:

$Q=a_1v_1=a_xv_x$, $a_1^2v_1^2=a_x^2v_x^2$, and $\frac{a_1^2v_1^2}{2g}=\frac{a_x^2v_x^2}{2g}$

then:

$$\frac{v_x^2}{2g}=\left(\frac{a_1}{a_x}\right)^2\frac{v_1^2}{2g}$$

Equation (4) then can be written:

$$H_T=\frac{v_1^2}{2g}\left[\left(\frac{a_1}{a_6}\right)^2K_t+\left(\frac{a_1}{a_5}\right)^2\left(K_e+K_{b_5}+\frac{fL_5}{D_5}+K_{ex}\right)\right.$$
$$+\left(\frac{a_1}{a_4}\right)^2\left(\frac{fL_4}{D_4}-K_{ex}-K_c\right)+\left(\frac{a_1}{a_3}\right)^2(K_c+K_g+K_{ex})$$
$$+\left(\frac{fL_1}{D_1}-K_{ex}+K_{b_1}-K_c\right)$$
$$\left.+\left(\frac{a_1}{a_2}\right)^2(K_c+K_g+K_v)\right] \quad (5)$$

If the bracketed part of the expression is represented by K_L, the equation can be written:

$$H_T=K_L\frac{v_1^2}{2g} \tag{6}$$

Then:

$$Q=a_1\sqrt{\frac{2gH_T}{K_L}} \tag{7}$$

228. *Pressure Flow Losses in Conduits.*—(a) *General.*—Head losses in outlet works conduits are caused primarily by the frictional resistance to flow along the conduit side walls. Additional losses result from trashrack interferences, entrance contractions, contractions and expansions at gate installations, bends, gate and valve constrictions, and other interferences in the conduit. As with free-flow conduits, greater than average loss coefficients should be assumed for computing required conduit and component sizes, and smaller loss coefficients should be used for computing energies of flow at the outlet. The major contributing losses of a conduit or pipe system are discussed in the remainder of this section.

(b) *Friction Losses.*—For flow in large pipes, the Darcy-Weisbach formula is most often employed to determine the energy losses due to frictional resistances of the conduit. The loss of head is stated by the equation:

$$h_f=\frac{fL}{D}\frac{v^2}{2g} \tag{8}$$

where f is the friction loss coefficient. This coefficient varies with the conduit surface roughness and with the Reynolds number. The latter is a function of the diameter of the pipe, and the velocity, viscosity, and density of the fluid flowing through it. Data and procedures for evaluating the loss coefficient are presented in a Bureau of Reclamation engineering monograph [3]. Since f is not a fixed value, many engineers are unfamiliar with its variations and would rather use Manning's coefficient of roughness, n, which has been more widely defined. If the effect of the Reynolds number influence is neglected, and if the roughness factor in relation to the pipe size is assumed constant, the relation of f in the Darcy-Weisbach equation to n in the Manning equation will be:

$$f=\frac{116.5n^2}{r^{1/3}}=\frac{185n^2}{D^{1/3}} \tag{9}$$

Relationships between the Darcy-Weisbach and Manning's coefficients can be determined from figure B–7 (appendix B).

Where the conduit cross section is horseshoe or rectangular in shape, the Darcy-Weisbach formula does not apply because it is for circular pipes, and the Manning equation may be used to compute the friction losses. Manning's equation as applied to closed conduit flow is:

$$h_f=29.1n^2\frac{L}{r^{4/3}}\frac{v^2}{2g} \tag{10}$$

Maximum and minimum values of n which may be used to determine the conduit size and the energy of flow are as follows:

	Maximum value	*Minimum value*
Concrete pipe or cast in place conduit	0.014	0.008
Steel pipe with welded joints	.012	.008
Unlined rock tunnel	.035	.020

(c) *Trashrack Losses.*—Trashrack structures which consist of widely spaced structural members without rack bars will cause very little head loss, and trashrack losses in such a case might be neglected in computing conduit losses. When the trash structure consists of racks of bars, the loss will depend on the bar thickness, depth, and spacing. An average approximation can be obtained [2] from the empirical equation:

$$K_t=1.45-0.45\frac{a_n}{a_g}-\left(\frac{a_n}{a_g}\right)^2 \tag{11}$$

where:

K_t = the trashrack loss coefficient,
a_n = the net area through the rack bars, and
a_g = the gross area of the racks and supports.

Where maximum loss values are desired, assume that 50 percent of the rack area is clogged. This will result in twice the velocity through the trashrack. For minimum trashrack losses, assume no clogging of the openings when computing the loss coefficient, or neglect the loss entirely.

(d) *Entrance Losses.*—The loss of head at the entrance of a conduit is comparable to the loss in a short tube or in a sluice. If H is the head producing the discharge, C is the coefficient of discharge, and a is the area, the discharge

Q is equal to $Ca\sqrt{2gH}$

and the velocity

$$v \text{ is equal to } C\sqrt{2gH}.$$

Or,

$$H=\frac{1}{C^2}\frac{v^2}{2g} \tag{12}$$

Since H is the sum of the velocity head h_v and the head lost at the entrance h_e, equation (12) may be written:

$$\frac{v^2}{2g}+h_e=\frac{1}{C^2}\frac{v^2}{2g} \text{ or } h_e=\left(\frac{1}{C^2}-1\right)\frac{v^2}{2g}$$

Then:

$$K_e=\left(\frac{1}{C^2}-1\right) \tag{13}$$

Coefficients of discharge for square sluice entrances are shown on figure 249. Coefficients of discharge and loss coefficients for typical entrances for conduits, as given in various texts and technical papers, are listed in table 30.

TABLE 30.—*Coefficients of discharge and loss coefficients for conduit entrances*

	Coefficient C			Loss coefficient K_e		
	Maximum	Minimum	Average	Maximum	Minimum	Average
(a) Gate in thin wall—unsuppressed contraction	0.70	0.60	0.63	1.80	1.00	1.50
(b) Gate in thin wall—bottom and sides suppressed.	.81	.68	.70	1 20	0.50	1.00
(c) Gate in thin wall—corners rounded	.95	71	.82	1.00	.10	0.50
(d) Square-cornered entrances	.85	.77	.82	.70	.40	.50
(e) Slightly rounded entrances	.92	.79	.90	.60	.18	.23
(f) Fully rounded entrances $\frac{r}{D}\geq 0.15$	.96	.88	.95	.27	.08	.10
(g) Circular bellmouth entrances	98	.95	.98	10	.04	.05
(h) Square bellmouth entrances	97	.91	.93	.20	.07	.16
(i) Inward projecting entrances	.80	.72	.75	.93	.56	.80

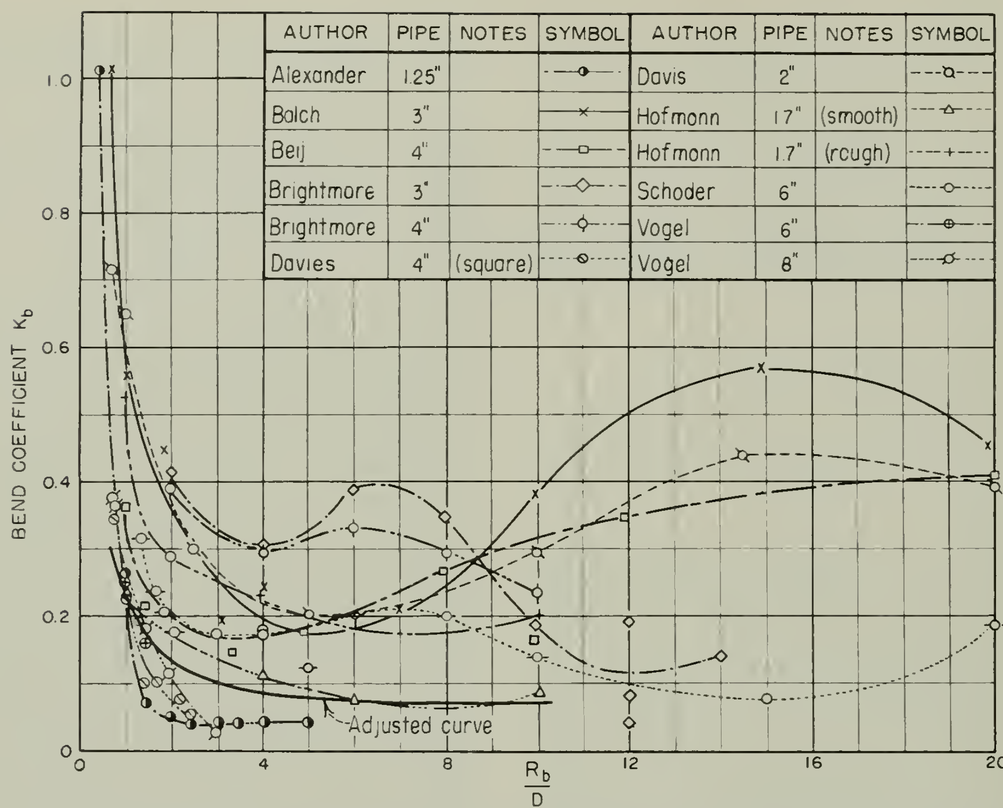
(e) *Bend Losses.*—Bend losses in closed conduits in excess of those due to friction loss through the length of the bend are a function of the bend radius, pipe diameter, and the angle through which the bend turns. Although experimental data on bend losses in large pipes are meager, the loss can be related to those determined for smaller pipe. Figure 251(A) shows the coefficients found by various investigators for 90° bends for various ratios of radius of bend to diameter of pipe, and an adjusted curve assumed to be suitable for large pipes. Figure 251(B) indicates the correction factor to be applied to the values indicated in figure 251(A) for other than 90° bends. The value of the loss coefficient, K_b, for various values of $\frac{R_b}{D}$ can be applied directly for circular conduits; for rectangular conduits D is taken as the height of the section in the plane of the bend.

(f) *Transition Losses.*—Head losses in gradual contractions or expansions in a conduit can be considered in relation to the increase or decrease in velocity head, and will vary according to the rate of change of the area and the length of the transition. For contractions the loss of head, h_c, will be approximately equal to $K_c\left(\frac{v_2^2}{2g}-\frac{v_1^2}{2g}\right)$, where K_c varies from 0.1 for gradual contractions to 0.5 for abrupt contractions. Where the flare angle does not exceed that indicated in section 229(b), the loss coefficient can be assumed as 0.1. For greater flare angles, the loss coefficient can be assumed to vary in a straight-line relationship to a maximum of 0.5 for a right angle contraction.

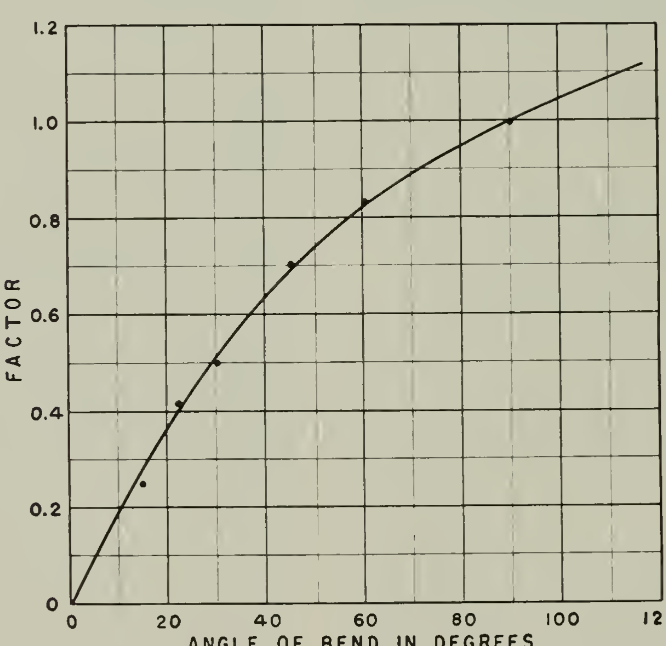
For expansions, the loss of head, h_{ex}, will be approximately equal to $K_{ex}\left(\frac{v_1^2}{2g}-\frac{v_2^2}{2g}\right)$, where K_{ex} is as follows:

Flare angle α	2°	5°	10°	12°	15°	20°	25°	30°	40°	50°	60°
K_{ex} [1]	0.03	0.04	0.08	0.10	0.16	0.31	0.40	0.49	0.60	0.67	0.72
K_{ex} [4]	02	.12	.16	.	.27	40	.55	.66	90	1.00	

(g) *Gate and Valve Losses.*—No gate loss need be assumed where a gate is mounted at the entrance to a conduit so that when wide open it does not interfere with the entrance flow conditions. Where a gate is mounted at either the upstream or downstream side of a thin headwall such that the sides and bottom of the jet are suppressed but the top is contracted, loss coefficients shown as item b in table 30 will apply. Where a gate is mounted in a conduit so that the floor, sides, and roof both upstream and downstream are continuous with the gate opening, only the losses due to the slot will need to be considered, for which a



(A) VARIATION OF BEND COEFFICIENT WITH RELATIVE RADIUS FOR 90° BENDS OF CIRCULAR CROSS SECTION, AS MEASURED BY VARIOUS INVESTIGATORS



(B) FACTORS FOR OTHER THAN 90° BENDS

Figure 251. Bend loss coefficients.

value of K_g not exceeding 0.1 might be assumed. For partly open gates, the coefficient of loss will depend on the top contraction; for smaller openings it will approach the value of 1.0 as shown for item b in table 30.

For wide open gate valves K_g will approximate 0.19. Similar to partly open gates, values of the loss coefficient will increase for smaller valve openings. Indicated loss coefficients for partly open gate valves are 1.15 for three-fourths open, 5.6 for one-half open, and 24.0 for one-fourth open. Average values of K_g for butterfly valves in the wide open position are about 0.15; values vary between 0.1 and 0.5, depending on the thickness of the gate leaf in relation to the gross area. Losses in spherical valves are negligible.

(h) *Exit Losses.*—No recovery of velocity head will occur where the release from a pressure conduit freely discharges, or is submerged or supported on a downstream floor. The velocity head loss coefficient, K_e, in these instances is equal to 1.0. When a diverging tube is provided at the end of a conduit, recovery of a portion of the velocity head will be obtained if the tube expands gradually and if the end of the tube is submerged. The velocity head loss coefficient will then be reduced from the value of 1.0 by the degree of velocity head recovery. If a_1 is the area at the beginning of the diverging tube and a_2 is the area at the end of the tube, K_e is equal to $\left(\frac{a_1}{a_2}\right)^2$.

229. *Transition Shapes.*—(a) *Entrances.*—To minimize head losses and to avoid zones where cavitation pressures can develop, the entrance to a pressure conduit should be streamlined to provide smooth, gradual changes in the flow. To obtain the best inlet efficiency, the shape of the entrance should simulate that of a jet discharging into air. As with the nappe-shaped weir, the entrance shape should guide and support the jet with minimum interference until it is contracted to the dimensions of the counduit. If the entrance curve is too sharp or too short, subatmospheric pressure areas which may induce cavitation will develop. A bellmouth entrance which conforms to or slightly encroaches upon the free-jet profile will provide the best entrance shape. For a circular entrance, this shape can be approximated by an elliptical entrance curve represented by the equation

$$\frac{x^2}{(0.5D)^2}+\frac{y^2}{(0.15D)^2}=1 \qquad (14)$$

where x and y are coordinates whose x-x axis is parallel to and $0.65D$ from the conduit centerline and whose y-y axis is normal to the conduit-centerline and $0.5D$ downstream from the entrance face. The factor D is the diameter of the conduit at the end of the entrance transition.

The jet issuing from a square or rectangular opening is not as easily defined as one issuing from a circular opening; the top and bottom curves may differ from the side curves both in length and curvature. Consequently, it is more difficult to determine a transition which will eliminate subatmospheric pressures. An elliptical curved entrance which will tend to minimize the negative pressure effects is defined by the equation:

$$\frac{x^2}{D^2}+\frac{y^2}{(0.33D)^2}=1 \qquad (15)$$

where D is the vertical height of the conduit for defining the top and bottom curves, and is the horizontal width of the conduit for defining the side curves. The major and minor axes are positioned similarly to those indicated for the circular bellmouth.

For a rectangular entrance with the bottom placed even with the upstream floor and with curved guide piers at each side of the entrance opening, both the bottom and side contractions will be suppressed and a sharper contraction will take place at the top of the opening. For this condition the top contraction curve is defined by the equation:

$$\frac{x^2}{D^2}+\frac{y^2}{(0.67D)^2}=1 \qquad (16)$$

where D is the vertical height of the conduit downstream from the entrance shape.

(b) *Contractions and Expansions.*—To minimize head losses and to avoid cavitation tendencies along the conduit surfaces, contraction and expansion transitions to and from gate control sections in a pressure conduit should be gradual. For contractions, the maximum convergent angle

should not exceed that indicated by the relationship:

$$\tan \alpha = \frac{1}{U} \qquad (17)$$

where:

α = the angle of the conduit wall surfaces with respect to its centerline, and

U = an arbitrary parameter $\frac{v}{\sqrt{gD}}$.

The values of v and D are the average of the velocities and diameters at the beginning and end of the transition.

Expansions should be more gradual than contractions because of the danger of cavitation where sharp changes in the side walls occur. Furthermore, as has been indicated in section 228(f), loss coefficients for expansions increase rapidly after the flare angle exceeds about 10°. Expansions should be based on the relationship:

$$\tan \alpha = \frac{1}{2U} \qquad (18)$$

The notations are the same as for equation (17). For usual installations, the flare angle should not exceed about 10°.

The criteria for establishing maximum contraction and expansion angles for conduits flowing partly full are the same as those for open channel flow, as given in section 197(b).

(c) *Exit Transitions.*—When a circular conduit flowing partly full empties into a chute, the transition from the circular section to one with a flat bottom can be made in the open channel downstream from the conduit portal, or it can be made within the conduit so that the bottom will be flat at the portal section. Ordinarily, the transition is made by gradually decreasing the circular quadrants from full radius at the upstream end of the transition to zero at the downstream end. For usual installations the length of the transition can be related to the exit velocity. An empirical rule which will give a satisfactory transition is:

$$L \text{ (in feet)} = \frac{vD}{5} \qquad (19)$$

where:

v = the exit velocity in feet per second, and
D = the conduit diameter in feet.

Downstream from a free-flow conduit the chute sections, including the transition into a stilling basin, will be governed by open channel flow criteria. Floor curvatures and maximum flare angles should be determined by equations (19) and (21), respectively, of chapter VIII. To reduce the length of the open channel portion from the conduit portal to the stilling basin, the beginning of the flare and of the convex curve may be located inside the conduit. This transition may be combined with the transition of the bottom shape.

In certain instances, as illustrated on figure 241, Crane Prairie Dam, and figure 243, Scofield and Newton Dams, an adverse slope and a hump have been employed immediately downstream from the portal to permit more rapid widening of the channel before it enters the basin. No firm criteria have been established for the design of these devices; the details were determined by model tests. Certain inherent disadvantages to this type of design are: (1) Care must be taken to avoid a hump of such height that back pressure will cause a hydraulic jump to occur inside the conduit, (2) the floor section at the hump must be made structurally sufficient to withstand the large dynamic forces resulting from impingement of the flow on the rising floor, (3) during periods of no flow a pond which can freeze during the winter is formed in the conduit unless provision is made to drain the sump, and (4) access into the downstream conduit is difficult unless drainage is provided. Depending on tailwater conditions, pumping may be required to provide drainage.

230. *Terminal Structures.*—(a) *General.*—Deflector buckets, hydraulic jump basins, and impact type stilling basins are suitable terminal structures for free-flow conduits, when appropriately used. These structures are commonly used in conjunction with spillways, and their hydraulic designs are discussed in part E of chapter VIII. Other types of stilling devices employed more often with outlet works than with spillways are plunge basins and stilling wells. The hydraulic designs of these structures are discussed in this section.

The hydraulic jump stilling basin, however, is most often used for energy dissipation of outlet works discharges. Where flow emerges from the outlet in the form of a free jet, as will be the case with valve-controlled outlets of pressure conduits,

it must be directed onto the transition floor approaching the basin so it will become uniformly distributed before entering the basin. Otherwise proper dissipation of energy will not be obtained.

To evaluate the energy which must be dissipated by the stilling device, the losses through the outlet system should be minimized, as discussed in sections 226 and 228(b). The specific energy immediately downstream from a gate or valve control will equal the exit velocity head based on minimum losses through the pressure system, as measured above the outflowing water surface. If specific energies have not been computed, approximate basin depths can be obtained from figure 208, as discussed in section 199(d).

(b) *Plunge Basins.*—Where the outlet conduit ends with a flip bucket or where flows issue from a downstream control valve or freely discharging pipe, a riprap- or concrete-lined trapezoidal plunge basin might be utilized. Such a basin should be employed only where the jet discharges into the air and then plunges downward into the basin. Tests have shown that if the angle of impingement is too flat the jet will ride and skip across the surface at high velocity. This will cause waves and eddies in the basin sufficient to erode the side slopes, and there will be high exit velocities.

As indicated in section 203, no fixed criteria have yet been established for plunge basins which will provide satisfactory dissipation for all heads, discharges, and incoming jet conditions. However, criteria that were established for several small outlet works plunge basins which have operated reasonably satisfactorily are herewith presented for use only as a preliminary guide to determine approximate basin geometry. The general arrangement of this basin is represented on figure 252. The basin depths were made about one-fifth of the difference in elevation between maximum reservoir water surfaces and maximum tailwater levels. The minimum bottom widths were made the width of the incoming jet, or the width required to limit the average velocity at the end of the basin to about 3 feet per second, whichever was greater.

(c) *Stilling Wells.*—Stilling well designs as described in section 223 are illustrated on figures 253 and 254. The well dimensions and performance criteria for these designs were established from model tests, and general criteria for such designs applicable to various conditions were not determined. The hydraulic stilling action in these devices results from turbulence and diffusion of the incoming high-energy flow into the water bulk in the well, and successful stilling is aided materially by special fillets and diffuser blocks incorporated along the sides and in the corners of the well. The net area of the well is generally selected by limiting the average rising velocity to between 1 and 3 feet per second. The total depth of the well will be dictated by the energy of the incoming flow which must be dissipated, and by the effectiveness of the diffuser blocks and fillets in diffusing the rising flow. Basins with similar criteria can be patterned after those illustrated in the figures. Basins for considerably different conditions should be model tested.

231. *Chart for Estimating Pressure Conduit Sizes.*—Figure 255 is a nomograph for the solution of the equation for pipe flow. By use of this figure, the required conduit diameter can be determined for a given length and gross head from reservoir water surface to the end of the pipe. As will be noted from the equation, an average total head loss coefficient of 0.5 has been assumed for all contributing losses except friction. It will also be noted that a value of n of 0.013 was used in determining friction losses. Adjustments for other values of n can be made by using a compensated value for L determined as shown on the figure.

To illustrate the use of the nomograph, assume the value of n to be 0.013, an available gross head of 5 feet, a conduit length of 300 feet, and a required discharge of 300 second-feet. First, find the intersection of the discharge and length on the right-hand portion of the figure. Lay a straightedge extending from this intersection to the 5-foot mark on the gross head scale. Where the straightedge intersects the 300-foot length on the left portion of the diagram, read the diameter of 5.75 feet. Similarly, for a 40-foot gross head, a length of 300 feet, and a discharge of 300 second-feet, a 3.7-foot-diameter pipe is found to be needed.

If the size of an unlined tunnel having an estimated value of n of 0.0225 is desired for the same length and discharge as before, the adjusted length of conduit to be used will be equal to $\left(\frac{n}{0.013}\right)^2 L=\left(\frac{0.0225}{0.013}\right)^2 300=900$ feet. The required diameter for a 5-foot gross head is found to be

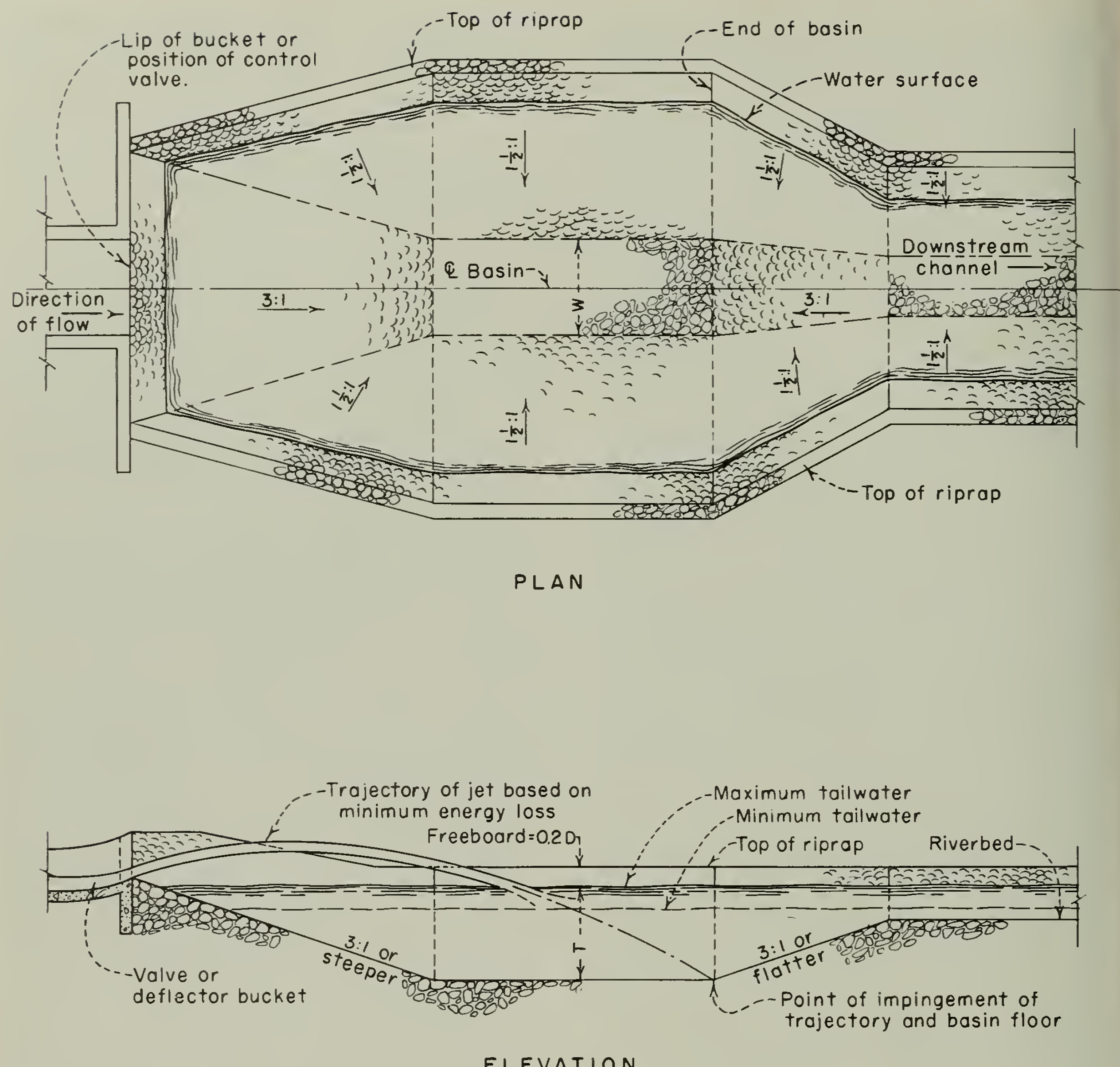


Figure 252. **Plunge basin energy dissipator.**

approximately 6.5 feet, from the chart. For a 40-foot head, the required diameter is found to be about 4.3 feet.

The chart can also be used to estimate the sizes of pipe in a compound system by considering each size separately and then adjusting for the recovery of velocity head. For example, assume an upstream conduit 200 feet long and 7 feet in diameter discharging 300 second-feet into a downstream pipe 300 feet long and 4 feet in diameter. On the basis of an n of 0.013, the gross head for a separate pipe of the dimensions of the downstream pipe will be about 29 feet. Similarly, the required head for a separate pipe of the dimensions of the upstream conduit will be about 2 feet. The velocity head, h_v, in the upstream pipe will be equal to:

$$\frac{1}{2g}\left(\frac{Q}{a}\right)^2=\frac{1}{64.4}\left[\frac{(4)\ (300)}{\pi\ (7^2)}\right]^2=0.95 \text{ feet.}$$

Then the gross head required will be 29 feet +2 feet −0.95 foot, or approximately 30 feet.

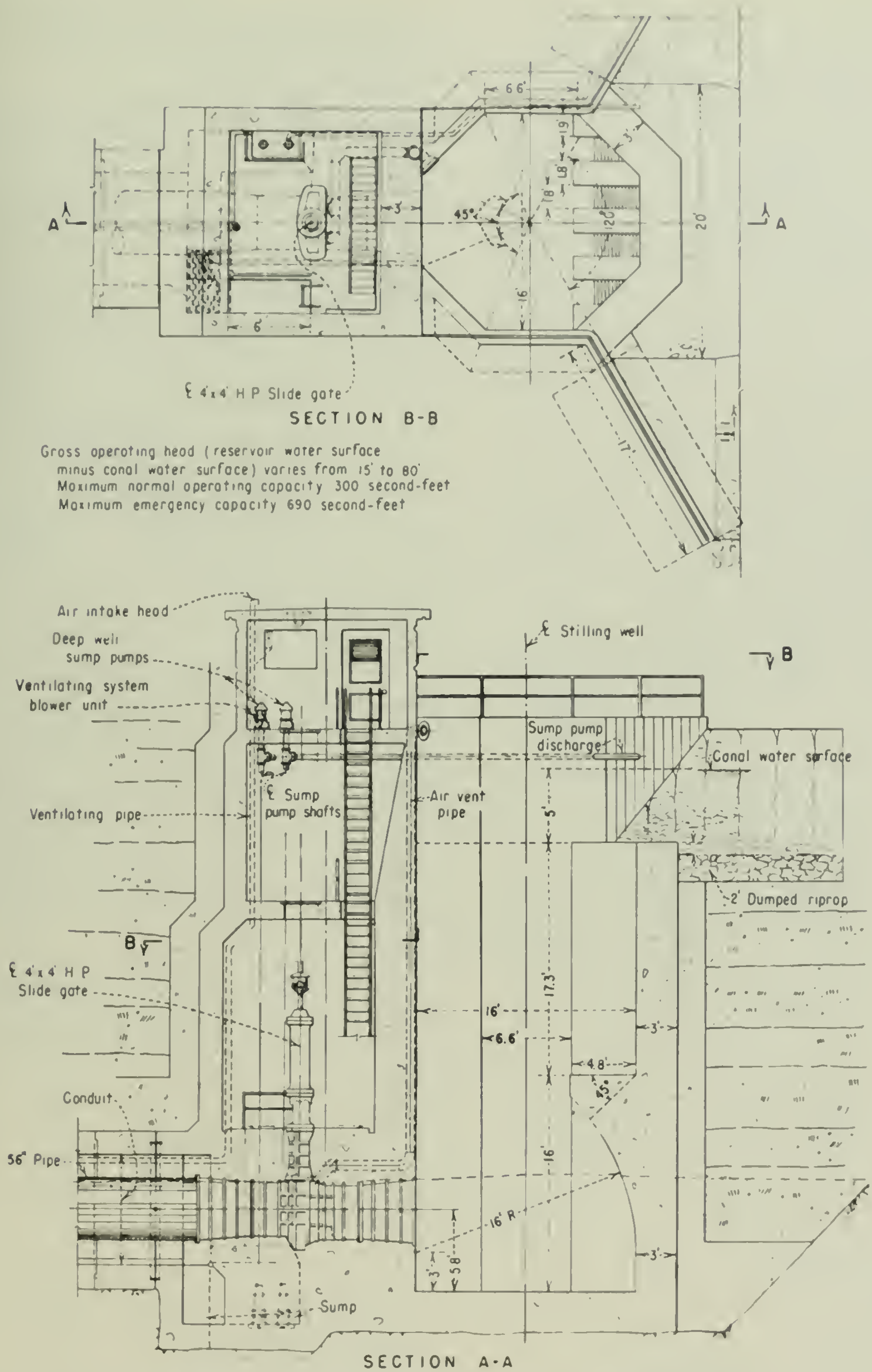


Figure 253. Riser well energy dissipator installation at Trenton Dam, Neb.

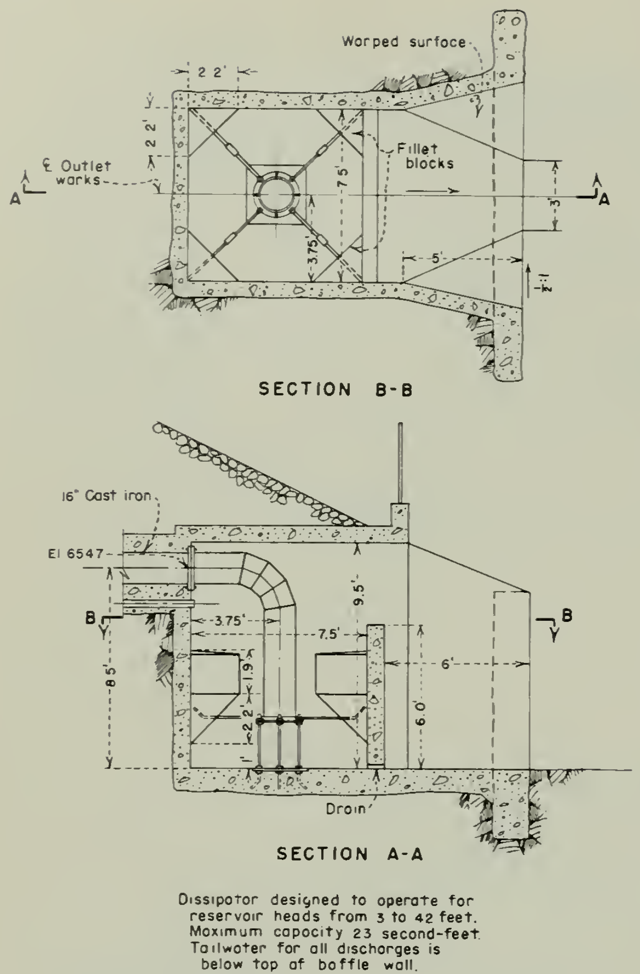


Figure 254. **Stilling well energy dissipator installation at Rattlesnake Dam, Colo.**

232. *Design Examples.*—To illustrate the procedures for hydraulic design of outlet works two examples are presented.

(a) *Example 1.*—The problem is to compute a discharge curve for the conduit for Wasco Dam shown on figure 154, and to check the stilling basin for the condition of design discharge. The outlet works is to discharge 300 second-feet with the reservoir at normal water surface. The solution is as follows:

First determine the total head needed to produce flow in terms of the component losses from equation (4), listing both maximum and minimum assumed losses and relating the loss coefficients to the area of the conduit barrel. These assumptions and computations are tabulated in table 31.

From equation (7), $Q=a_1\sqrt{\frac{2gH_T}{K_L}}$, for maximum loss conditions Q is equal to $20\sqrt{\frac{64.4H_T}{12.28}}=45.8\sqrt{H_T}$. For minimum loss conditions Q is equal to $20\sqrt{\frac{64.4H_T}{7.98}}=56.8\sqrt{H_T}$. Discharges based on the average for these two extremes will be represented approximately by the equation $Q=51\sqrt{H_T}$. A discharge curve for this relationship as shown on figure 154 can be computed if the value of H_T is determined. Since the jet issuing from the gate opening is supported, H_T will be measured from the normal water surface to the top of the jet.

The depth of water downstream from the gate opening may be estimated by use of the coefficient

TABLE 31.—*Computation of total loss coefficients—Example 1*

Element	Area of element, square feet	$\left(\frac{a_1}{a_x}\right)^2$	Loss	Coefficient symbol	Maximum loss considerations		Minimum loss considerations	
					Coefficient	$\left(\frac{a_1}{a_x}\right)^2\times$coefficient	Coefficient	$\left(\frac{a_1}{a_x}\right)^2\times$coefficient
Trashrack	Gross, 326; net, 272	0.005	Trashrack	[1] K_t	1.09	0.01	0.36	0.00
Entrance	20.0	1.00	Entrance	[2] K_e	.20	.20	.07	.07
Conduit	20.0	1.00	Friction	[3] K_f	.51	.51	.17	.17
			Transition	[4] K_c	.20	−.20	.10	−.10
Gate	9.0	4.9	do	K_c	.20	.98	.10	.49
			Gate	[5] K_g	1.20	5.88	.50	2.45
			Exit	K_v	1.00	4.90	1.00	4.90
Total loss coefficient, K_L						12.28		7.98

a_1 = area of conduit.
a_x = area of element.
[1] From equation (11):
For maximum loss, $K_t=1.45-0.45\left(\frac{136}{326}\right)-\left(\frac{136}{326}\right)^2=1.09$
For minimum loss, $K_t=1.45-0.45\left(\frac{272}{326}\right)-\left(\frac{272}{326}\right)^2=0.36$
[2] From table 30, item h.

[3] $K_f=29.1\,n^2\left(\frac{L}{r^{4/3}}\right)$. $L=101$ feet, $r=\frac{20}{18}=1.11$, $n_{max}=0.014$, $n_{min}=0.008$

[4] Because of the wet well immediately above the transition, assume a maximum K_c of 0.2 and a minimum of 0.1.

[5] From table 30, item b. Note that when both gates are wide open, the downstream gate will not be submerged because of the top contraction of the issuing stream through the upstream gate and therefore will not affect the flow.

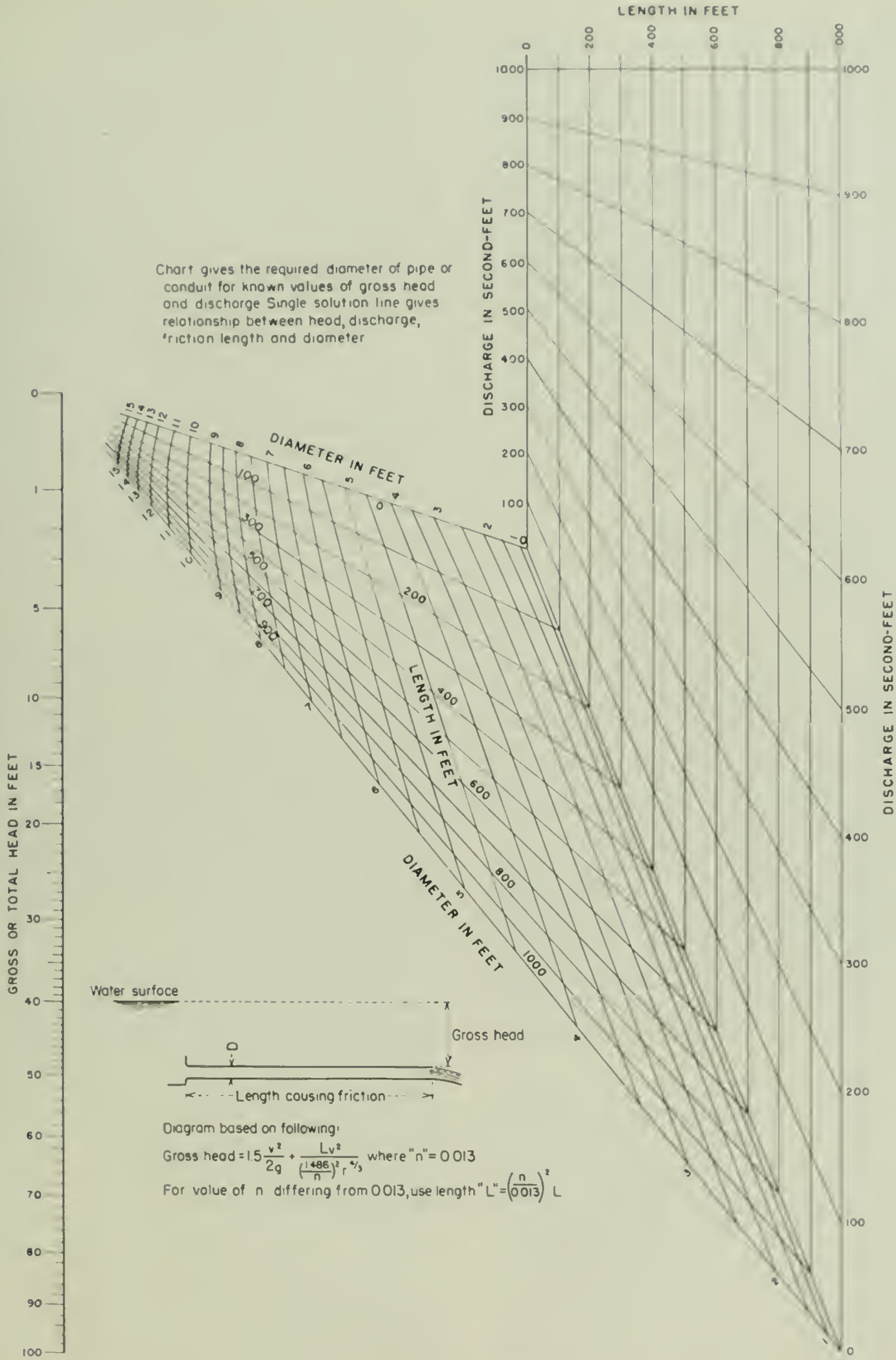


Figure 255. **Chart for solution of the discharge through large pipes flowing under pressure.**

of discharge for the gate, which in this case is an approximate measure of the top contraction. From item b of table 30, the maximum, minimum, and average values of the coefficient of discharge are 0.81, 0.68, and 0.70, respectively. The approximate depth of water downstream from the gate will be these values multiplied by the height of the gate (3.0 feet), or 2.4, 2.0, and 2.1 feet, respectively. Values of H_T for maximum, minimum, and average losses are found by subtracting the elevation of the corresponding downstream water surfaces from the normal water surface elevation of 3514.4, as follows:

H_T (maximum losses) $=3514.4-(3478.5+2.0)=$ 33.9 feet.

H_T (minimum losses) $=3514.4-(3478.5+2.4)=$ 33.5 feet.

H_T (average losses) $=3514.4-(3478.5+2.1)=$ 33.8 feet.

The corresponding discharges are:

Q (maximum losses) $=45.8\ (33.9)^{1/2}=267$ second-feet.

Q (minimum losses) $=56.8\ (33.5)^{1/2}=329$ second-feet.

Q (average losses) $=51\ (33.8)^{1/2}=297$ second-feet.

The computed discharge for average losses corresponds closely with the 300-second-foot discharge for which the outlet works was to be designed; therefore that portion of the system which flows under pressure can be considered to meet the hydraulic design requirements.

In order to analyze the downstream free-flow portion of the outlet works, the hydraulic gradient immediately below the gate must be determined. The hydraulic gradients for both maximum and minimum losses are computed, and the average hydraulic gradient thereby obtained is used with the average discharge which has already been computed.

With minimum losses, the discharge is equal to 329 second-feet at normal reservoir water surface elevation 3514.4. If the gate and exit loss coefficients in the last column of table 31 are subtracted from K_L, the losses upstream from the gate are found to be $0.63h_{v_1}$. For a discharge of 329 second-feet through the conduit barrel, the velocity will be $\frac{329}{20}=16.5$ feet per second and the velocity head will be 4.2 feet. The loss up to the gate is equal to $(0.63)\ (4.2)=2.7$ feet, indicating the hydraulic gradient just upstream from the gates to be at elevation $3514.4-2.7=3511.7$. This provides a net head to the center of the gate of $3511.7-3480.0=31.7$ feet.

For the usual assumption that the coefficient of velocity through a standard orifice is 0.98, the velocity at the contracted section downstream from the gate is equal to $0.98\sqrt{(64.4)(31.7)}=44.3$ feet per second ($h_v=30.9$ feet). The area of the contracted section is equal to $\frac{333}{44.3}=7.5$ square feet. For the 4-foot-wide free-flow downstream conduit, the depth will be $\frac{7.5}{4}=1.9$ feet, thus providing a gradient which is $d+h_v$ or $1.9+30.9=32.8$ feet above the invert of the conduit and establishing the hydraulic gradient at elevation 3511.3.

The discharge at normal water surface assuming maximum loss conditions is 267 second-feet. Following the same procedure as above, the gradient is found to be at elevation 3509.3. The average elevation of the hydraulic gradient is $\frac{3511.3+3509.3}{2}$ $=3510$.

The next step is to compute the flow through the downstream free-flow conduit for the design discharge of 300 second-feet, which previous computations have shown to be approximately equal to the discharge through the pressure portion of the outlet works with average loss conditions. Here again, the losses should be maximized and minimized to determine extreme conditions at the downstream portal. Computations can be tabulated as shown in tables 32 and 33. (For procedure see sec. 196.)

For the stilling basin design, from figure 206 for a value of F_1 of 9.4 and a d_1 of 0.68, as indicated from table 32, the required tailwater depth will be 8.7 feet, which closely matches an actual tailwater depth of 9 feet. From table 33 the depth indicated at the downstream portal will be about 3 feet, indicating that the conduit will be about 0.6 full. This condition of flow should provide ample air space to forestall sealing from splash or wave action. The design of the free-flow portion of the conduit is therefore satisfactory for the design discharge of 300 second-feet.

(b) *Example 2.*—The problem is to design an outlet works system similar in layout to that shown for Soldier Canyon Dam on figure 244, capable of discharging 100 second-feet at reservoir

Table 32. *Hydraulic computation for free-flow portion of conduit—Example 1*

Minimum Losses

[n=0.008, Q=300 second-feet]

Station	ΔL	Trial d	Width	a	v	h_v	r	$r^{2/3}$	s	$\frac{s_1+s_2}{2}$	Δh_L	$\Sigma\Delta h_L$	$d_2+h_{v_2}+\Sigma\Delta h_L$	Invert elevation	Datum gradient	Remarks
3+38		1.71	4.0	6.84	43.9	29.9	0.92	0.95	0.062				31.5	3178.[illegible]	3[illegible]	
3+93	55	1.9	4.0	7.6	39.5	24.2	.97	.98	.047	.054	3.0	3.0	29.1	31[illegible].8	3[illegible].8	Too low
		1.8		7.2	41.7	27.0	.95	.97	.054	.058	3.2	3.2	31.9		3[illegible].7	OK
4+18	55	1.9	4.0	7.6	39.5	24.2	.97	.98	.047	.050	2.8	6.0	32.1	3177.0	3[illegible].1	OK
4+68	20	.7	10.0	7.0	42.9	28.5	.61	.72	.103	.075	1.5	7.5	36.7	3171.0	3[illegible].7	Too low
		.68		6.8	44.1	30.2	.60	.71	.112	.080	1.6	7.6	38.5		3[illegible].5	OK

Then at the upstream end of the hydraulic jump basin, d_1=0.68, v_1=44.1 and $F_1=\frac{v_1}{\sqrt{gd_1}}=9.4$

Table 33.—*Hydraulic computations for free-flow portion of conduit—Example 1*

Maximum Losses

[n=0.018, Q=300 second-feet]

Station	ΔL	Trial d	Width	a	v	h_v	r	$r^{2/3}$	s	$\frac{s_1+s_2}{2}$	Δh_L	$\Sigma\Delta h_L$	$d_2+h_{v_2}+\Sigma\Delta h_L$	Invert elevation	Datum gradient	Remarks
3+38		1.71	4.0	6.84	43.9	29.9	0.92	0.95	0.31				31.6	3178.5	3510.0	
3+93	55	2.0	4.0	8.0	37.5	21.8	1.0	1.0	.21	0.26	14.3	14.3	38.1	3177.8	3515.9	Too high
		2.2		8.8	34.1	18.0	1.05	1.03	.16	.23	12.7	12.7	32.9		3510.7	OK
4+18	55	2.5	4.0	10.0	30.0	14.0	1.11	1.07	.12	.14	7.7	20.3	36.8	3177.0	3513.8	Too high.
		2.9		11.6	25.9	10.4	1.18	1.12	.08	.12	6.6	19.3	32.6		3509.6	OK

elevation 100.0. The centerline of the downstream control valve is at elevation 75.0, and the sill of the drop inlet intake is at elevation 85.0. The length of the pressure conduit upstream from the emergency closure gate valve is 300 feet, including the vertical length and the length around the bend at the inlet. The length of the downstream pressure pipe is 200 feet. The solution is as follows:

First, an evaluation of the approximate size of the conduit can be obtained by the use of the chart shown on figure 255 and the procedure illustrated in section 231. A pipe approximately 2.75 feet in diameter will be required for the entire length of 500 feet for the available head of 25 feet, if the losses other than for friction can be approximated at 0.5 h_v. Because of the installation of gates or valves in the system, these losses will likely exceed that amount and a somewhat larger pipe may be required.

For the indicated pipe size, a 30-inch butterfly control valve might be considered. Assuming that K_g is 0.5, the discharge coefficient of the valve is equal to $\sqrt{\frac{1}{1.5}}=0.818$, say 0.8. The head needed to discharge 100 second-feet will then be $h_g=\frac{1}{2g}\left(\frac{Q}{aC}\right)^2=\frac{1}{64.4}\left[\frac{100}{(4.91)(0.8)}\right]^2=10.1$ feet. This will leave about 15 feet for all other losses.

Next, consider the pipe size downstream from the gate chamber. A size equal to the 2.75-foot uniform diameter determined previously for the entire length might be used. The loss, h_f, through the 200 feet of length will then equal $\frac{fL}{D}\frac{v^2}{2g}$; or for an n of 0.012, from figure B-7 (appendix B), f=0.0192, and $h_f=\left[\frac{(0.0192)(200)}{2.75}\right]\left[\frac{100}{5.94}\right]^2\left[\frac{1}{64.4}\right]=6.2$ feet. This loss plus the required head for the valve discharge will leave about 8.7 feet for upstream and other losses.

Next, select the conduit size upstream from the pipe. Assuming a 3.5-foot-diameter conduit with an n of 0.014, f=0.024; and the loss through the 300-foot length is $h_f=\left[\frac{(0.024)(300)}{3.5}\right]\left[\frac{100}{9.62}\right]^2\left[\frac{1}{64.4}\right]=3.4$ feet. This loss plus the pipe and

regulating valve losses of 6.2 and 10.1, respectively, total approximately 20 feet, leaving about 5 feet for other losses. This seems reasonable enough to warrant evaluation.

Assuming, then, a 3.5-foot-diameter upstream conduit, a 24-inch emergency gate valve, a 33-inch downstream pipe, and a 30-inch regulating valve, a detailed analysis of the losses can be made. The losses will be based on the maximum loss coefficients as discussed previously. Table 34 shows the results in tabular form.

Then from equation (7), $Q=a_1\sqrt{\frac{2gH_T}{K_L}}$, for a value of K_L of 6.02 (5.82), Q is equal to $5.94\sqrt{\frac{64.4H_T}{6.02}}$ (5.82) $=19.4$ (19.8) $\sqrt{H_T}$; or for a 25-foot head, $Q=97$ (99) second-feet. This value is slightly less than the design requirement, and one or more of the elements must be enlarged to reduce the total loss. If the emergency gate size is increased to 30 inches, the area designated as 4 in the table will change from 3.14 to 4.91. Then the items f, g, and h, which total 1.40, will change to $\left(\frac{3.14}{4.91}\right)^2$ (1.40), or 0.57, and the total value of K_L will be reduced to 5.19 (4.99). Then $Q=5.94\sqrt{\frac{64.4H_T}{5.19}}$ (4.99) $=20.9$ (21.3) $\sqrt{H_T}$; or for a head of 25 feet, $Q=105$ (106) second-feet.

TABLE 34.—*Computation of total loss coefficient—Example 2*

Element	Designated area subscript	Area, square feet	$\left(\frac{a_1}{a_x}\right)^2$	Item	Loss	Loss symbol	Loss coefficient	$\left(\frac{a_1}{a_x}\right)^2$ times loss coefficient
Trashrack	2	Gross, 60; [1] net, 25	0.06	a	Trashrack	[2] K_t	1.09	0.06
Entrance	3	9.62	.38	b	Entrance	[3] K_e	.10	.04
Upstream conduit	3	9.62	.38	c	Bend	[4] K_b	.10	.04
		9.62	.38	d	Friction	[5] K_f	2.06	.78
		9.62	.38	e	Contraction	K_c	.10	—.04
Gate valve	4	3.14	3.58	f	----do	K_c	.10	.36
		3.14	3.58	g	Gate valve	K_g	.19	.68
		3.14	3.58	h	Expansion	K_{ex}	.10	.36
Downstream pipe	1	5.94	1.0	i	----do	K_{ex}	.10	—.10
		5.94	1.0	j	Friction	[6] K_f	1.40	1.40
		5.94	1.0	k	Contraction	K_c	.10	÷10 .10
Regulating valve	2	4.91	1.46	l	----do	K_c	.10	.15
		4.91	1.46	m	Valve	K_g	.50	.73
		4.91	1.46	n	Exit	K_v	1.00	1.46
Total loss coefficient, K_L								6.02 5.82

a_1=area of conduit.
a_x=area of element.

[1] Assuming trashracks designed for 2 feet per second velocity and 50 percent clogged, gross area=net area×1.2=60.

[2] $K_t=1.45-0.45\left(\frac{25}{60}\right)-\left(\frac{25}{60}\right)^2=1.09.$

[3] From table 30, item g.

[4] From fig. 251.

[5] $K_f=\frac{fL}{D}$, $D=3.5$, $n=0.014$, f from fig. B-7 (appendix B)=0.024

$$K_f=\frac{(0.024)(300)}{3.5}=2.06$$

[6] $K_f=\frac{fL}{D}$, $D=2.75$, $n=0.012$, f from fig. B-7 (appendix B)=0.0192

$$K_f=\frac{(0.0192)(200)}{2.75}=1.40$$

D. STRUCTURAL DESIGN DETAILS

233. *General.*—The same types of structures may be used for either spillways or outlet works. Spillways utilize open channels more often than do outlet works, and the structural design details for open channels are therefore discussed in part G of chapter VIII. The details of the design of walls, open channel linings, and floors, discussed as spillway structures, are applicable to these structures when used for outlet works. Also, the headworks of open channel outlet works are similar to gated crest structures for spillways as regards structural design details.

On the other hand, closed conduit waterways are more commonly used for outlet works than they are for spillways, and the design details of closed conduits are therefore discussed in this

chapter. The design details are the same in either case.

A closed conduit waterway might be a cast-in-place cut-and-cover culvert or conduit, a precast or prefabricated pipe, or a tunnel bored through the abutment. Waterways for a spillway will most often be free flowing, while those for outlet works may either flow full under pressure or partly full. The security of earthfill and rockfill dams depends to a large degree on the safety of the spillway and outlet structures, especially when conduits pass through the embankment. In those cases where all or part of a conduit is under internal pressure due to reservoir head, any leakage or failure of the conduit may cause openings through the dam which may gradually be enlarged until partial or complete failure results. There is also the danger of seepage along the contact surfaces between the conduit and the earthfill, which may result in serious damage. A third danger is the possibility of structural collapse of the conduit which would be almost certain to result in failure of an earthfill dam. These facts emphasize the importance of using durable materials, conservative design procedures, proper design details, and construction methods that will insure safe structures.

Replacement of a conduit through either an earthfill or rockfill dam is usually a difficult and expensive operation which can be avoided by the use of permanent material, such as cast iron for small size pipes and reinforced concrete cast-in-place conduit or precast concrete pipe for larger sizes. For small reservoirs with comparatively low heads, exceptions may be made only where the possible damage from failure is of little or no consequence. In such cases, where it is economically advantageous, the use of iron or steel pipe protected by galvanizing or bituminous coating or by some other rust-resisting treatment may be justifiable. It should be recognized that buried steel pipe is vulnerable to deterioration from electrolytic action, even where rust-resisting treatments or protective coatings are provided. Limited service with a possibility of eventual failure must therefore be expected. The use of iron or steel pipe as a watertight liner in a concrete conduit is permissible. However, unencased metal pipe should not be used if loss of life or serious damage to property will result from failure of the dam which might follow deterioration of the pipe.

Conduit joints must be made watertight to prevent leakage into the surrounding embankment. Joints of concrete cast-in-place conduits must be sealed with waterstops, and rubber-gasketed joints must be used for precast concrete pipe. For metal pipe, couplings are required which will remain watertight after movement or settlement of the pipe. Corrugated metal pipe with riveted seams should not be used for conduits unless the seams are welded or adequately calked and sealed with durable materials. If used for pressure conduits, corrugated metal pipe should be pretested for watertightness with pressures equal to twice the operating head.

When the outlet conduit consists of prefabricated pipe, whether of reinforced concrete, cast iron, or steel, the methods of bedding the pipe and backfilling around it should be such as to insure, insofar as possible, against unequal settlement and to secure the most uniform possible distribution of load on the foundation. When filling around these structures, extreme care should be taken to secure tight contact between the fill and the conduit surface and to obtain proper densities of the earthfill material. (See sec. E–4, app. E.) This is important not only for the prevention of seepage along the conduit, but also to insure that the fill develops a lateral restraint on the structure which will prevent excessive stresses in the conduit shell.

When the outlet consists of precast reinforced concrete or metal pipe, it should be set carefully on a good foundation and well bedded in concrete, as shown on figure C–2 (app. C). The concrete bedding not only aids in distributing the conduit load on the foundation, but also guarantees against uncompacted zones and void spaces under the pipe which could induce leakage along the undersurface of the structure. Void spaces or inadequate compaction of impervious materials at the invert of pipes have been the cause of numerous failures of small earthfill dams. The practice of supporting pipes on piers or collars without a concrete bedding should be avoided, because the greater foundation reaction at the concentrated support points will cause unequal stress distribution in the pipe. Furthermore, if the foundation settles from under the conduit between piers, the unsupported con-

duit will sag and crack. If the conduit is sufficiently strong to sustain the fill load, the earth shrinking away from the underside will leave voids which will permit the free passage of water.

Details of design for cut-and-cover conduits are discussed in section 235.

234. *Tunnel Details.*—Linings are provided in tunnel waterways for both hydraulic and structural reasons. The smooth boundary surfaces reduce frictional resistance and permit a smaller diameter tunnel for a required capacity. Lining also is used to prevent saturation of the surrounding ground by seepage. Structural lining is used to support the tunnel walls against raveling or yielding ground.

The thickness of lining needed to form smooth surfaces or to reduce seepage is the minimum which will avert cracking from expansion and contraction due to temperature change. For ordinary linings where reasonably stable ground is encountered and where a minimum of tunnel support is required, an average lining thickness between ¾ and 1 inch per foot of tunnel diameter is ordinarily used. The minimum thickness usually provided is 6 inches. Yielding ground or areas through water-bearing strata may require thicker linings to resist external loads and hydrostatic pressures. A full circular lining is the most efficient shape to withstand such external loads.

Where the tunnel lining is to be reinforced, it must be made sufficiently thick both to accommodate the reinforcement mat and to provide sufficient room for placing the concrete in the confined space behind the forms. A minimum thickness of 8 inches is suggested for tunnel linings with a single layer of reinforcement. Where two layers of reinforcement are required, a minimum thickness of 12 inches may be desirable.

The portions of a tunnel which must be reinforced and the amount of reinforcement required depends on the physical features of the tunnel and many geological factors. For a free-flow tunnel, reinforcement may be required to resist external loads due to unstable ground or to grout or water pressures. Pressure tunnels with high hydrostatic loads must have lining reinforced sufficiently to withstand bursting where inadequate cover or unstable supporting rock prevails.

A suggested general guide for determining reinforcement requirements in tunnels is as follows:

(1) A pressure tunnel should ordinarily be reinforced whenever the depth of cover is less than about 1.5 times the unbalanced internal pressure head. For determining the required reinforcement, the external pressure is assumed to vary from full reservoir head at the upstream end of the tunnel to zero pressure at the control when it changes to a free-flow tunnel. The reinforcement should be sufficient to withstand bursting pressures without considering support from the surrounding rock.

(2) The transition of a pressure tunnel to a free-flow tunnel should be specially reinforced to prevent excessive cracking which would permit leakage from the pressure portion of the tunnel to enter behind the lining of the free-flow portion. Reinforcement for the pressure portion for a distance upstream from the junction equal to five times the diameter of the tunnel should be based on full internal hydrostatic head with no allowance for restraint from the surrounding rock. The free-flow portion of the tunnel should be reinforced for a distance downstream from the junction equal to twice the tunnel diameter, assuming that hydrostatic head equal to the internal head just upstream from the junction can be exerted on only a semicircular portion of the outside of the tunnel lining, which would cause an unbalanced external load.

(3) A nominal amount of both longitudinal and circumferential reinforcement should be provided near the portals of both pressure and free-flow tunnels to resist loads resulting from loosened rock headings or from sloughing of the portal cuts. This reinforcement should extend back from the portal faces for a distance equal to twice the tunnel diameter.

(4) If in competent rock, the tunnel other than at the portals and at the transition from pressure to free-flow may be unreinforced where the depth of cover is more than 1.5 times the unbalanced internal pressure head. If in unstable ground, lining should be reinforced to support probable rock loadings. Methods of estimating loadings for tunnel supports as given in the publication "Rock Tunneling With Steel Supports" [5] can be used to estimate requirements for reinforced lining.

If unstable material is encountered in driving a tunnel, it is usually necessary to provide some means of supporting the tunnel roof and sides until the concrete lining has been placed. These supports may be either temporary or permanent. Temporary supports, usually timber ribs and lagging, are removed before the concrete lining is placed, while permanent supports are left in place and embedded in concrete. Permanent supports consist of steel ribs, steel-liner plates, or a combination of the two. Individual roof bolts or wire mesh lagging with roof bolts also may be utilized to stabilize the crown of the tunnel to prevent raveling or rock falls during the construction period.

Supporting ribs must be capable of holding up large blocks of material whose natural support was removed in excavating the tunnel. The lagging must be closely spaced where the material slacks off in small pieces; elsewhere it may be widely spaced or even omitted. Methods of assuming and computing the size of supports are given by Proctor and White [5]. Loadings will be based on the nature of the ground encountered, and unless the exact underground conditions are known beforehand, no definite rules for design of the ground-support system can be established. The required size and spacing of supports are usually determined by trial as the work progresses. In permanently supported sections of the tunnel, all spaces outside of the lagging or liner plates should be filled as completely and compactly as possible with clean gravel or rock spalls, and the spaces thoroughly filled with grout after the lining has been placed.

For tunnels through jointed rock or where seepage is to be minimized, the areas surrounding the tunnel are usually grouted both to solidify the material and to fill open fissures in the rock and the voids between the lining and the rock. This grouting is accomplished by drilling holes through the lining into the surrounding rock and then injecting grout under pressure as described in part C of appendix G. Permissible grouting pressures will depend on the nature of the surrounding ground and on the lining thickness. For small tunnels, rings of grout holes are spaced at about 20-foot centers, depending on the nature of the rock. Each ring consists of four grout holes distributed at about 90° around the periphery, with alternate rings placed on vertical and 45° axes.

Drainage holes are often provided in other than pressure tunnels to relieve external pressures caused by seepage along the outside of the tunnel lining. The drainage holes also are spaced at about 20-foot centers, at intermediate locations between the grout hole rings. At successive sections, one vertical hole is drilled near the crown alternating with two drilled horizontal holes, one in each side wall. In free-flow tunnels, drainage holes are provided only above the water surface, if flow through the tunnel is conveyed in a separate pipe, the horizontal holes are drilled near the invert.

Typical details for both pressure and free-flow tunnels for a small capacity outlet works are shown in figure 256.

235. *Cut-and-Cover Conduit Details.* (a) *General.* The design of a cut-and-cover conduit to be constructed through or under an earthfill embankment must include details which will provide for movement and settlement without excessive cracking or leakage. To obtain a safe structure, the following factors must be considered:

(1) Provide devices to minimize seepage along the contact of the conduit and the impervious embankment.

(2) Provide details to forestall cracking which might result in leakage of water into the fill surrounding a pressure conduit and to prevent piping of embankment material into a free-flow conduit.

(3) Select and treat foundation to minimize differential settlement which is a cause of cracking.

(4) Provide a structure to safely carry the loads to which the conduit will be subjected.

Selection of designs and details to accomplish the above purposes is discussed in this section.

(b) *Cutoff Collars.*—Foundation preparation and compaction around conduits must be equivalent to foundation preparation for the dam and to compaction of the impervious earthfill. Projecting fins or cutoff collars are provided to minimize seepage along the contact between the outside surface of the conduit and the embankment. These collars should be made of reinforced concrete, generally from 2 to 3 feet high, 12 to 18 inches wide, and spaced from 7 to 10 times their

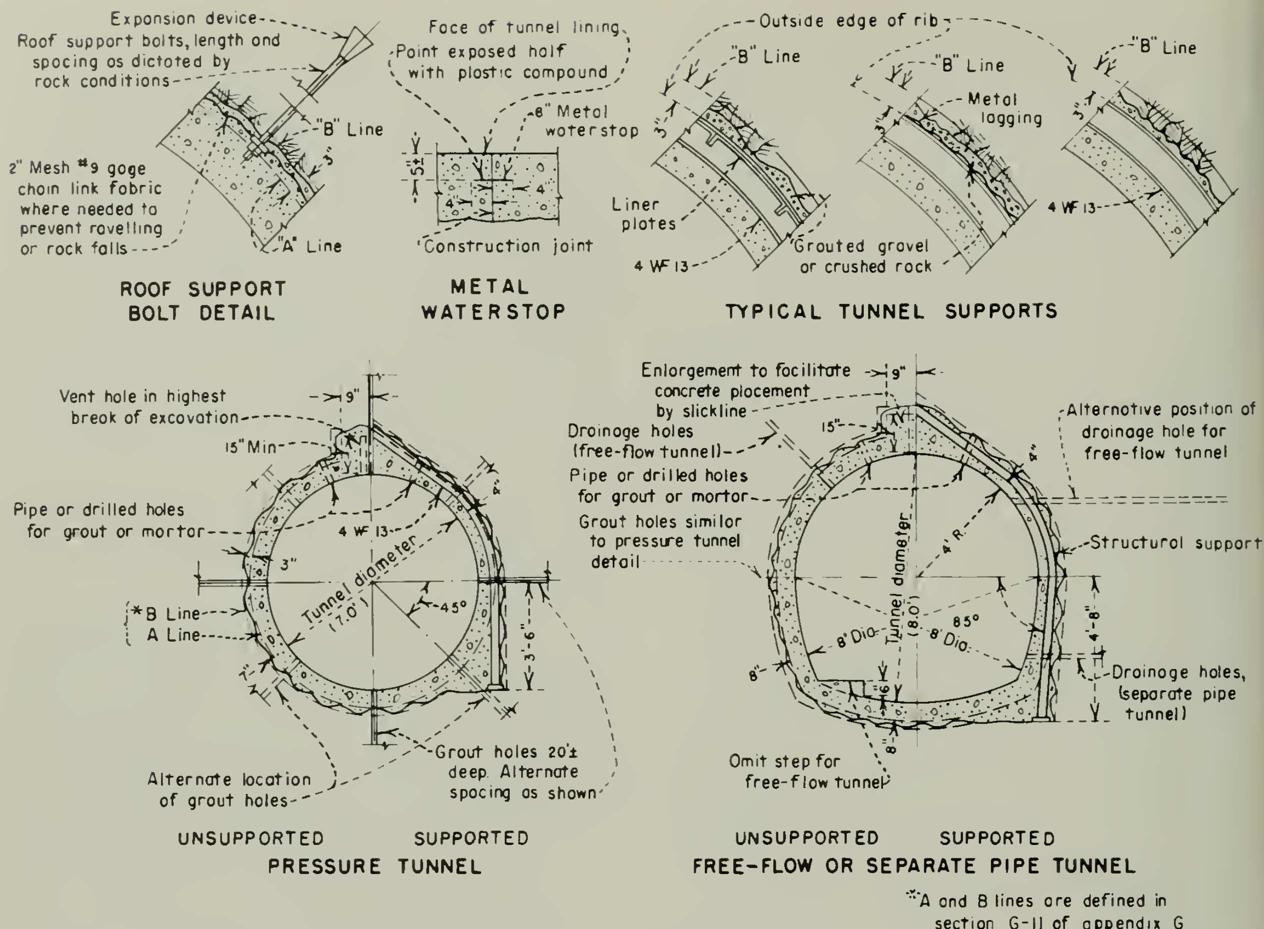


Figure 256. **Typical tunnel details.**

height along that portion of the conduit which lies within the impervious zone of the dam. The length of the percolation path along the contact is thereby increased by 20 to 30 percent.

For a conduit on an earth foundation, the collar should completely encircle the conduit barrel. Where the foundation is sound rock, good contact along the base may be expected and the collars need extend only to be keyed into the rock foundation. The collars should be separated from the conduit to avoid introducing concentrated stresses into the conduit walls, which would alter the normal stress in the barrel. This is accomplished by constructing the collars with watertight fillers between the collars and the barrel. The structural separation permits lateral slipping of the collar on the barrel, eliminates secondary stresses in the conduit which would otherwise be caused by the stiffening effect of the collars, and avoids the introduction of torsional stresses in the conduit if horizontal movement or displacement of the embankment should occur. The joint filler material can be several layers of graphite-coated paper if only slight movement is expected, or premolded bituminous fillers where greater movement is expected.

Although cutoff collars usually are located between joints in the conduit, there are cases where collars have been constructed to span the joints. When so located they also serve as watertight covers for the joints. Where the collar is not placed at a conduit joint or where it is placed over a joint which is restrained from movement by keyways or by reinforcement extending across it, the collar ordinarily will not be subjected to large lateral loadings. In such cases it will need to be only strong enough to resist the superimposed fill load. When a collar covers a joint designed to permit differential movement, either the collar must be designed sufficiently strong to

restrain such movement, or the collar must adjust to the movement without losing the watertight contact.

(c) *Conduit Joints.*—Conduits constructed on rock or competent earth foundations may be subjected only to small settlement and longitudinal movements. Cast-in-place conduits on such foundations can be made more or less monolithic and, except for movement caused by initial setting shrinkage and by temperature expansion and contration, should experience little cracking or joint opening. In such a design, major cracking is avoided by liberal use of longitudinal reinforcement placed across construction joints to form a continuous structure. During construction, the adjoining sections of the barrel are not constructed until after the major volume change in the first-placed section due to initial setting shrinkage has taken place. Waterstops of metal or rubber are placed across the joints to provide a watertight seal. Details of this type of joint construction are shown on figure 257.

Where considerable settlement and lateral or longitudinal adjustment of the foundation is expected, the conduit may be constructed as an articulated structure. The individual portions of the structure must be free to move without causing uncontrolled cracking which would permit leakage through the conduit walls. For such designs the reinforcement is not carried continuously across the joints, so that the individual sections are free to move longitudinally. Waterstops are provided to prevent leakage through the joints. Differential lateral displacement of the conduit sections at the joints is ordinarily restrained by a bell-and-spigot joint or by a reinforced collar encircling a plain joint. Rubber-gasketed joints similar to those shown on figure 258 can be adopted for joining individual lengths of concrete pipes. These joints can be used in cast-in-place conduits by embedding short sections of precast pipe which contain the joint detail. Specifications for pipe and pipe joints indicated on figure 258 can be found in ASTM specifications, Designation C 361–57T.

(d) *Design Loads.*—Embankment loads on conduits may vary over a wide range depending on many factors relating to the foundation, method of bedding, flexibility or rigidity of the conduit; and to the soil characteristics of the embankment such as angle of internal friction, unit weight, homogeneity, consolidation properties, cohesiveness, and moisture content. All possible combinations of these various factors must be considered to evaluate their overall effect. The loads must be considered not only as they may occur during construction but also as they may be altered after embankment completion, reservoir loading, and embankment saturation.

The "Marston Theory" of embankment pressures is usually adopted for small conduits under relatively low fills. This theory is discussed in many bulletins published by the Iowa State College Experiment Station and is abstracted in various handbooks [6, 7] which contain bibliographies of publications dealing with this subject.

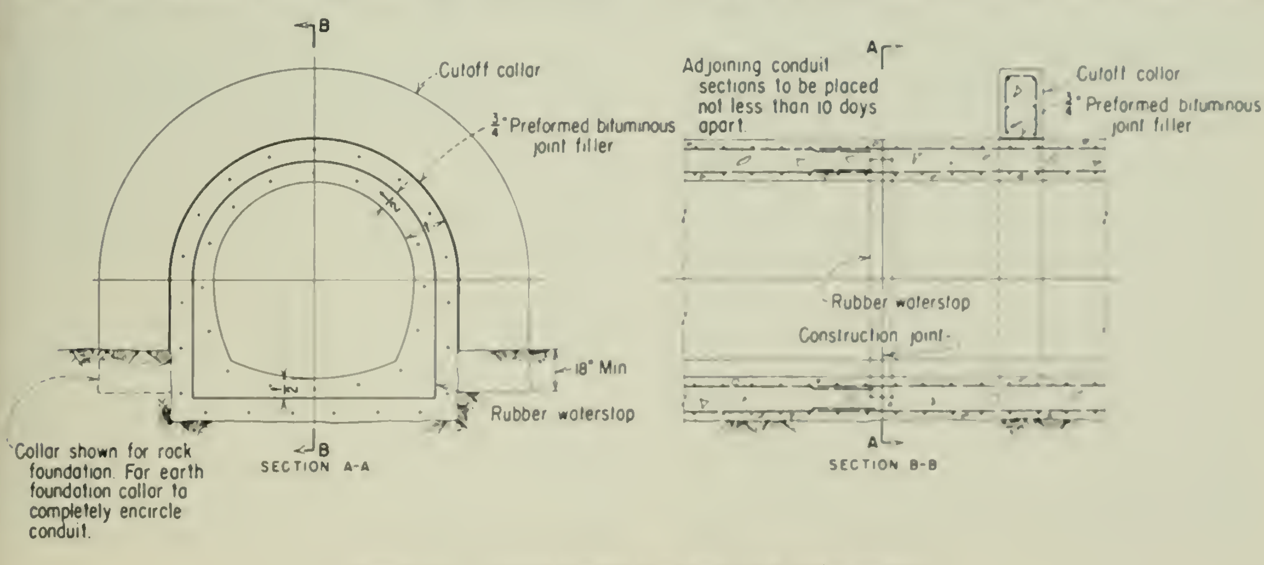


Figure 257. Typical conduit joint and cutoff collar details.

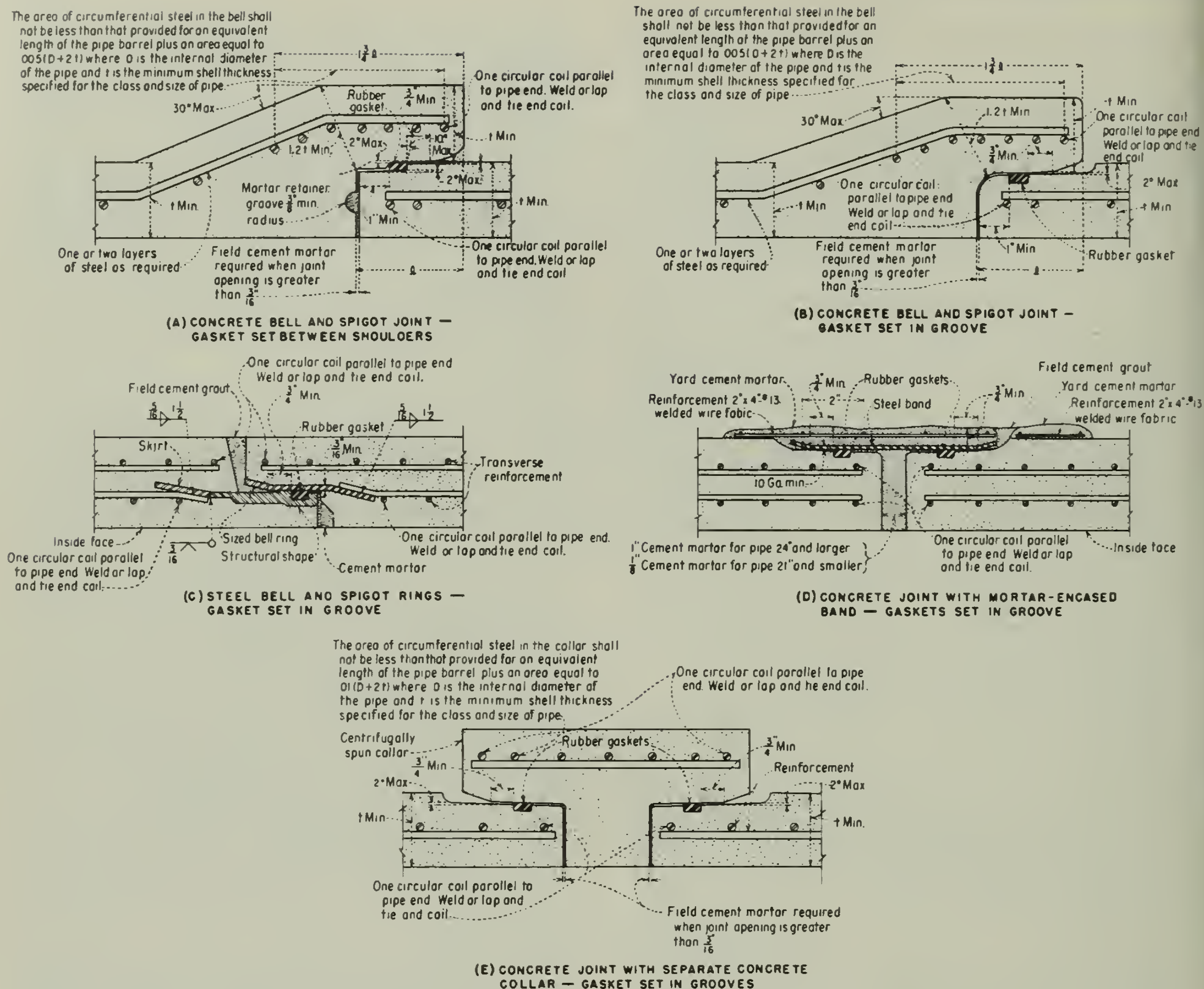


Figure 258. **Precast concrete pipe joint details.**

On the basis of the Marston theory, the vertical load on a conduit is considered to be a combination of the weight of the fill directly above the conduit and the frictional forces acting either upward or downward due to the adjacent fill. A settlement of adjacent fill greater than the overlying fill induces frictional forces acting downward which increase the resultant load on the conduit; a greater settlement immediately above the conduit will result in an arching condition which reduces the load on the conduit. Thus a conduit laid in trench excavated in a compact natural soil will practically never receive the full weight of the backfill above it, because of the development of arching action when the backfill starts to settle. On the other hand, if the conduit is placed so that it projects in whole or in part above the natural ground surface, the embankment load which may come upon it can in some cases be as much as 50 percent greater than the weight of the fill directly above it. The designs of precast concrete pressure pipe discussed in appendix C are based on loads as derived by the Marston theory.

For cast-in-place conduits under relatively high fills, where the conduit is placed in cut so that neither a full trench nor a complete projecting condition exists, a loading assumption which averages the extremes noted above is assumed. For this case the load on the conduit is assumed to be the weight of the column of fill directly above it. The load over that portion of a conduit

under the upstream part of the dam includes both the weight of the saturated fill and the weight of the reservoir water above the fill. The conduit barrel is designed on the basis of a given factor of safety, considering that the unit horizontal lateral load on the conduit is one-third of the unit vertical load. The design is then checked on the basis of a reduced factor of safety considering no horizontal lateral load exists. The vertical reaction of the base of the conduit is taken equal to the vertical load plus the weight of the conduit. On an earth foundation, the base reaction is assumed to be distributed uniformly across the width of the conduit; on a rock foundation it is assumed to be distributed triangularly, varying from twice the average unit reaction at the outside edges to zero at the center of the base. External hydrostatic pressures are assumed to act equally in all directions, vertically downward as an increased load, upward as uplift, and laterally on the sides of the conduit.

Procedures for designing concrete box culverts and circular conduits are comprehensively discussed in "Concrete Culverts and Conduits" [8]. Data for selecting a cast-in-place conduit design based on design procedures using Beggs Deformeter coefficients [9] are included in appendix C. These data list required conduit thickness and reinforcement requirements for various fill heights and hydrostatic loading conditions. Also included in appendix C are data for selecting precast concrete pipe for use as conduits under limited fill loads.

236. *Details of Typical Structures.* Figures 259 through 266 show arrangements for outlet works intakes, shafts, and stilling basins constructed at various small Bureau of Reclamation dams. These are presented as examples which can be used as guides in the design of similar structures.

E. BIBLIOGRAPHY

237. *Bibliography.*

[1] King, W. H., "Handbook of Hydraulics," fourth edition, McGraw-Hill Book Co., Inc., New York, N.Y., 1954.

[2] Creager, W. P., and Justin, J. D., "Hydroelectric Handbook," second edition, John Wiley & Sons, Inc., New York, N.Y., 1950.

[3] Bradley, J. N., and Thompson, L. R., "Friction Factors for Large Conduits Flowing Full," Engineering Monograph No. 7, U.S. Department of the Interior, Bureau of Reclamation, March 1951.

[4] Rouse, Hunter, "Engineering Hydraulics," John Wiley & Sons, Inc., New York, N.Y., 1950.

[5] Procter, R. V., and White, T. L., "Rock Tunneling with Steel Supports," Commercial Shearing & Stamping Co., Youngstown, Ohio, 1946.

[6] "Concrete Pipe Handbook," American Concrete Pipe Association, 228 North La Salle Street, Chicago 1, Ill., 1951.

[7] "Handbook of Drainage and Construction Products," Armco Drainage & Metal Products, Inc., Middletown, Ohio, 1958.

[8] "Concrete Culverts and Conduits," Portland Cement Association, 33 West Grand Avenue, Chicago, Ill.

[9] Phillips, H. B., "Beggs Deformeter Stress Analysis of Single-Barrel Conduits," Engineering Monograph No. 14, U.S. Department of the Interior, Bureau of Reclamation, April 1952.

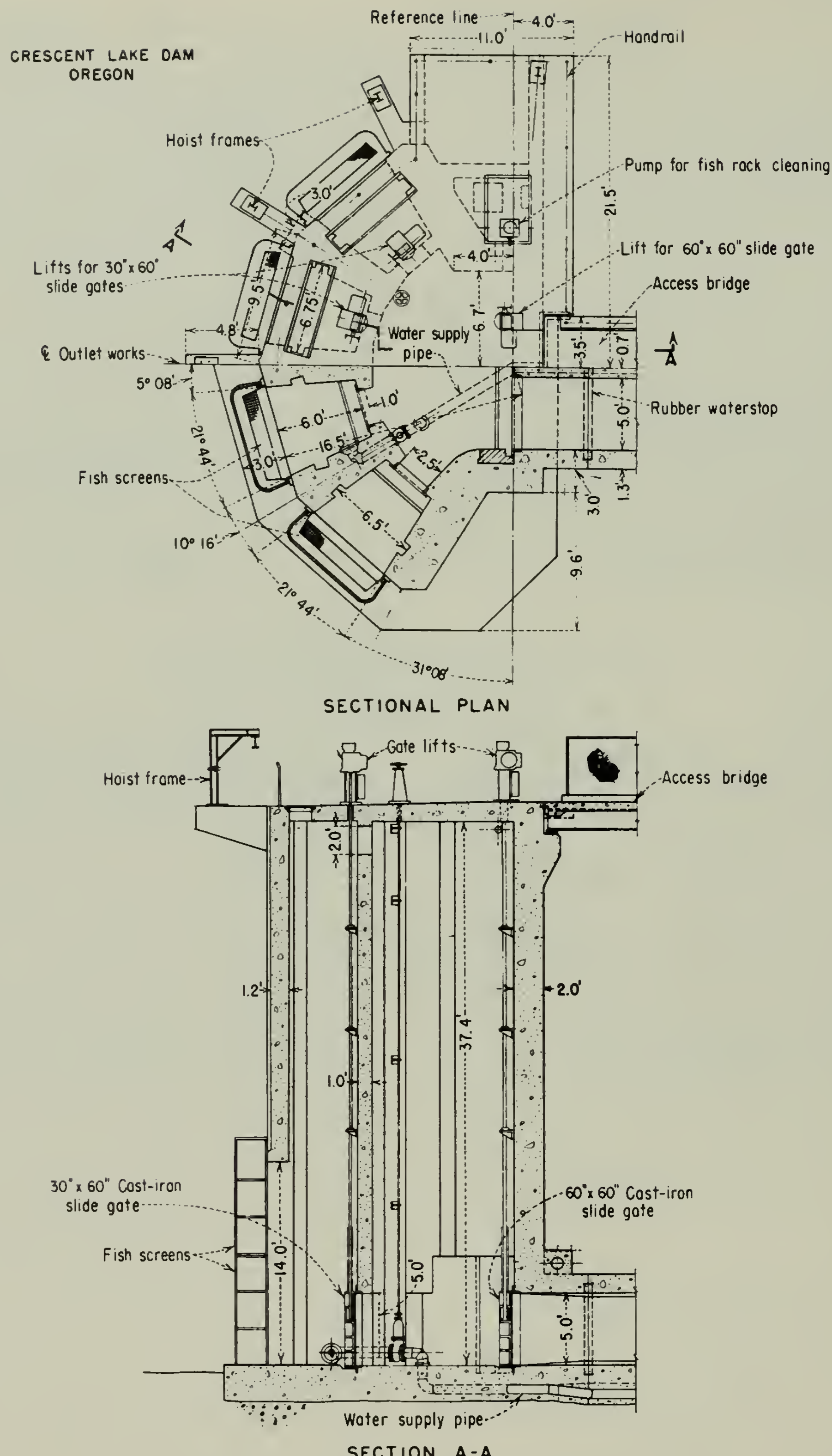


Figure 259. **Typical intake tower arrangement.**

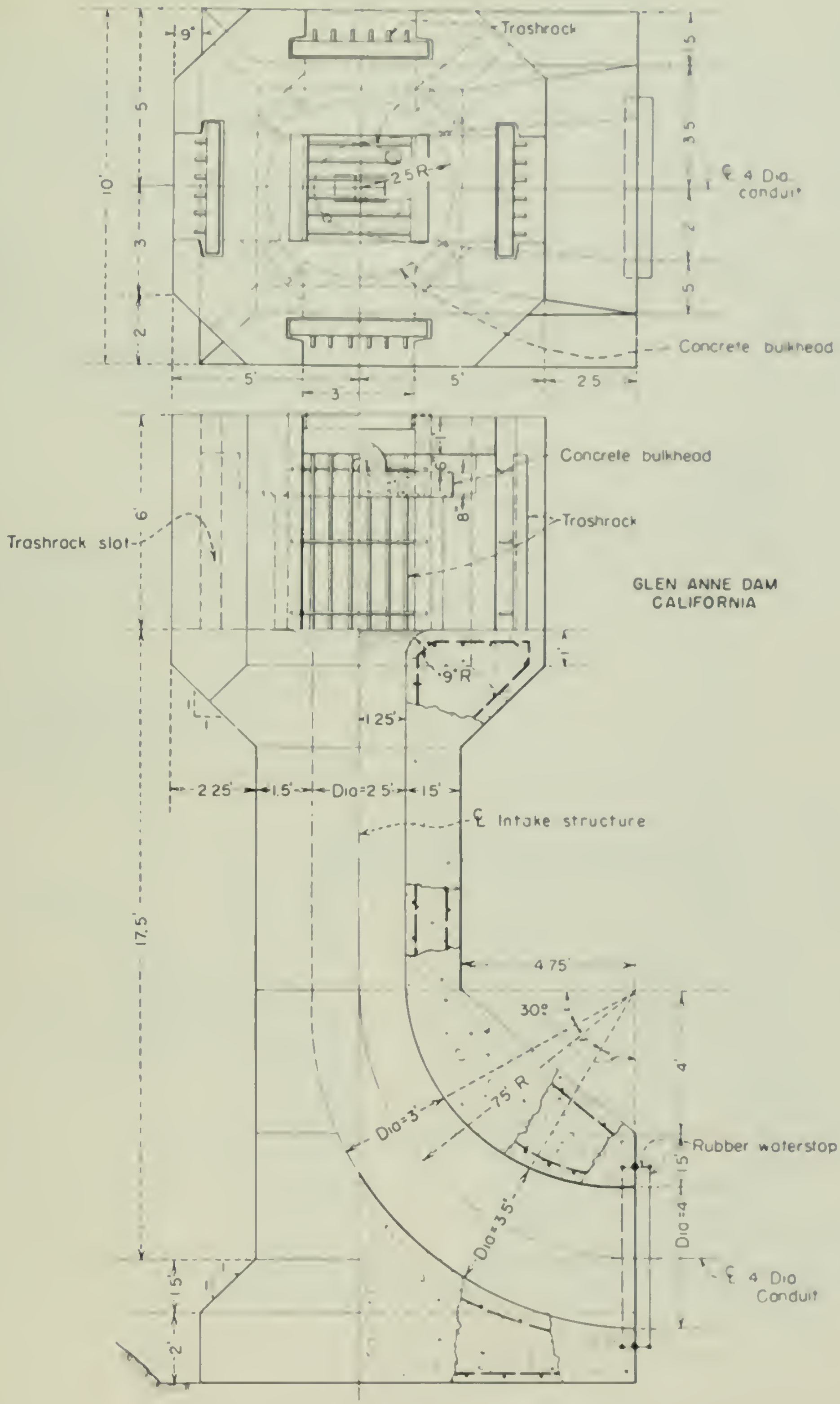
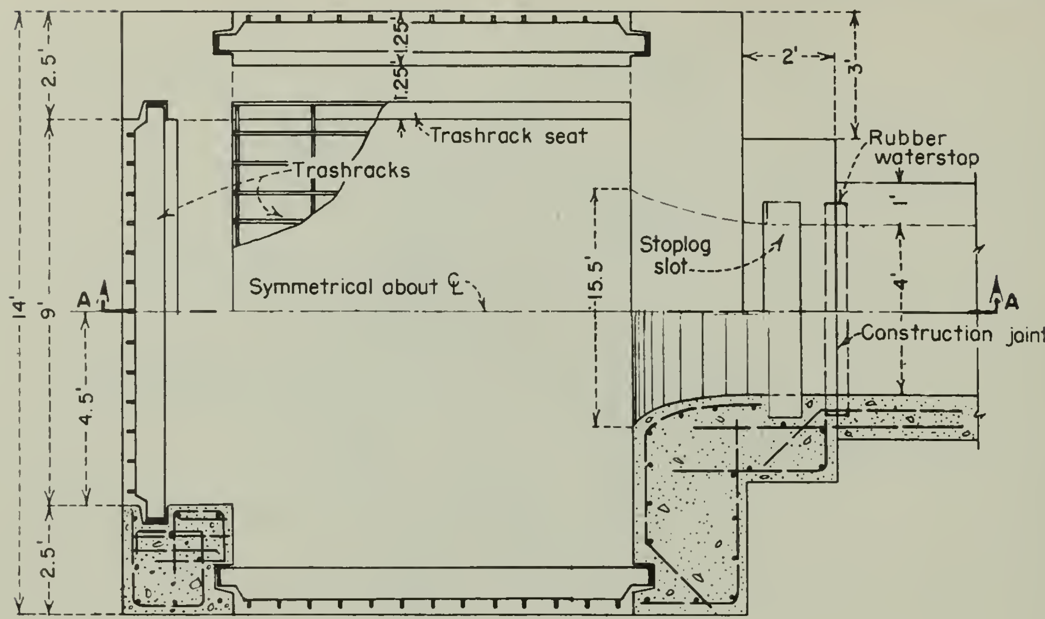
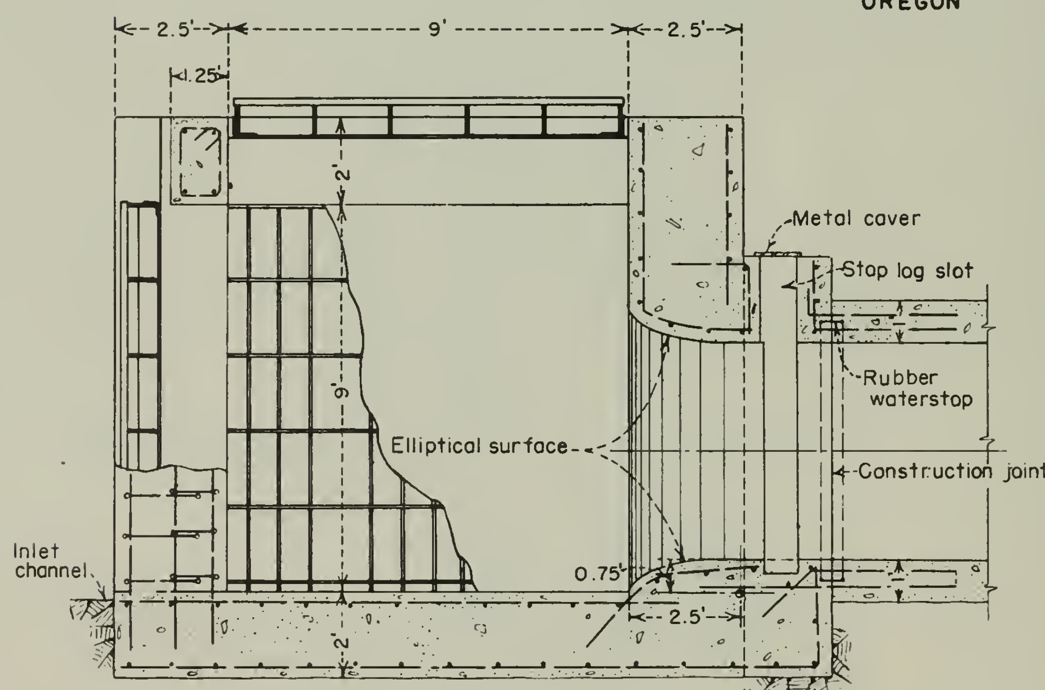


Figure 260. Typical drop inlet intake.



SECTIONAL PLAN



SECTION A–A

Figure 261. Typical trashracked box intake.

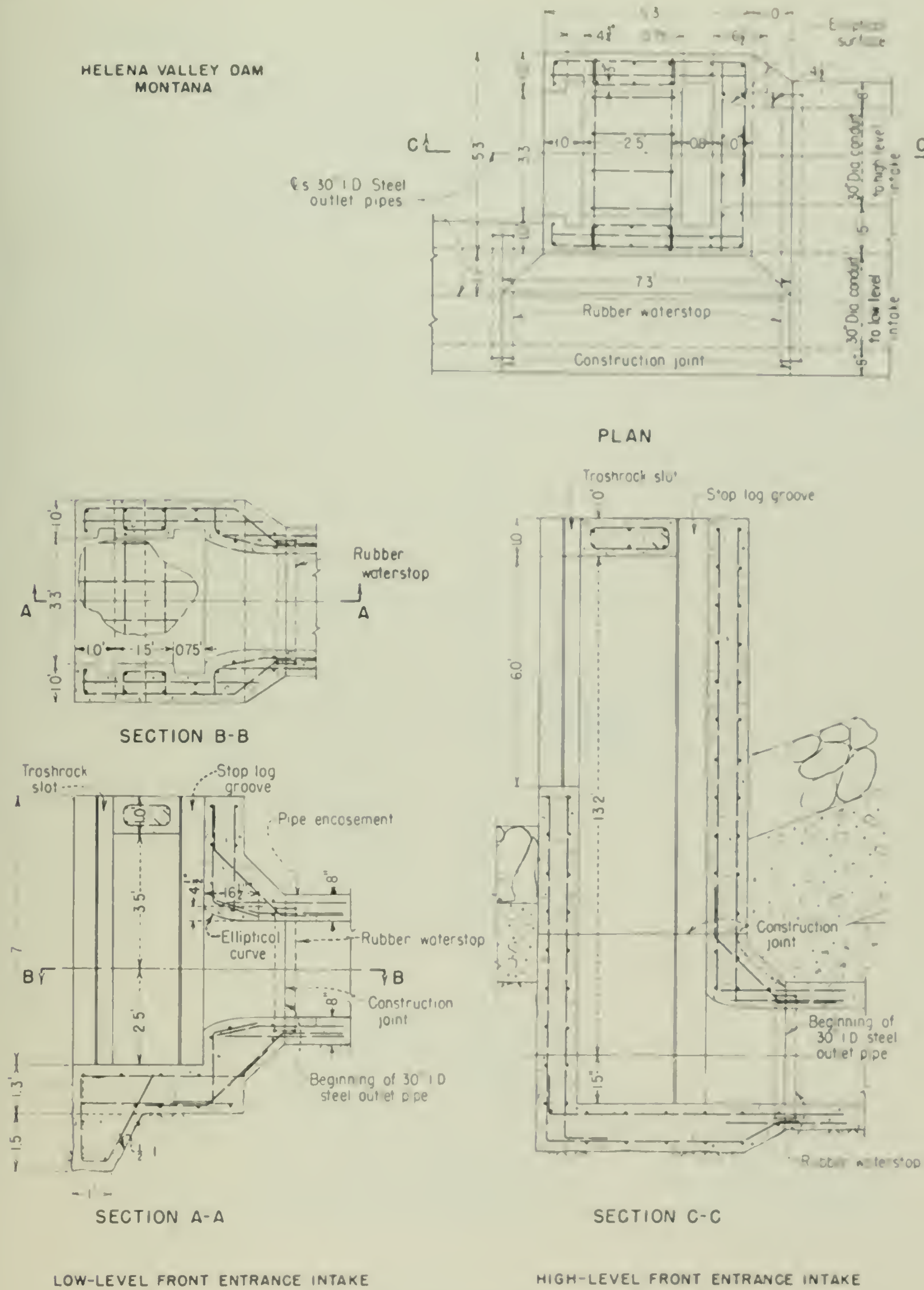


Figure 262. Typical front-entrance intake structures.

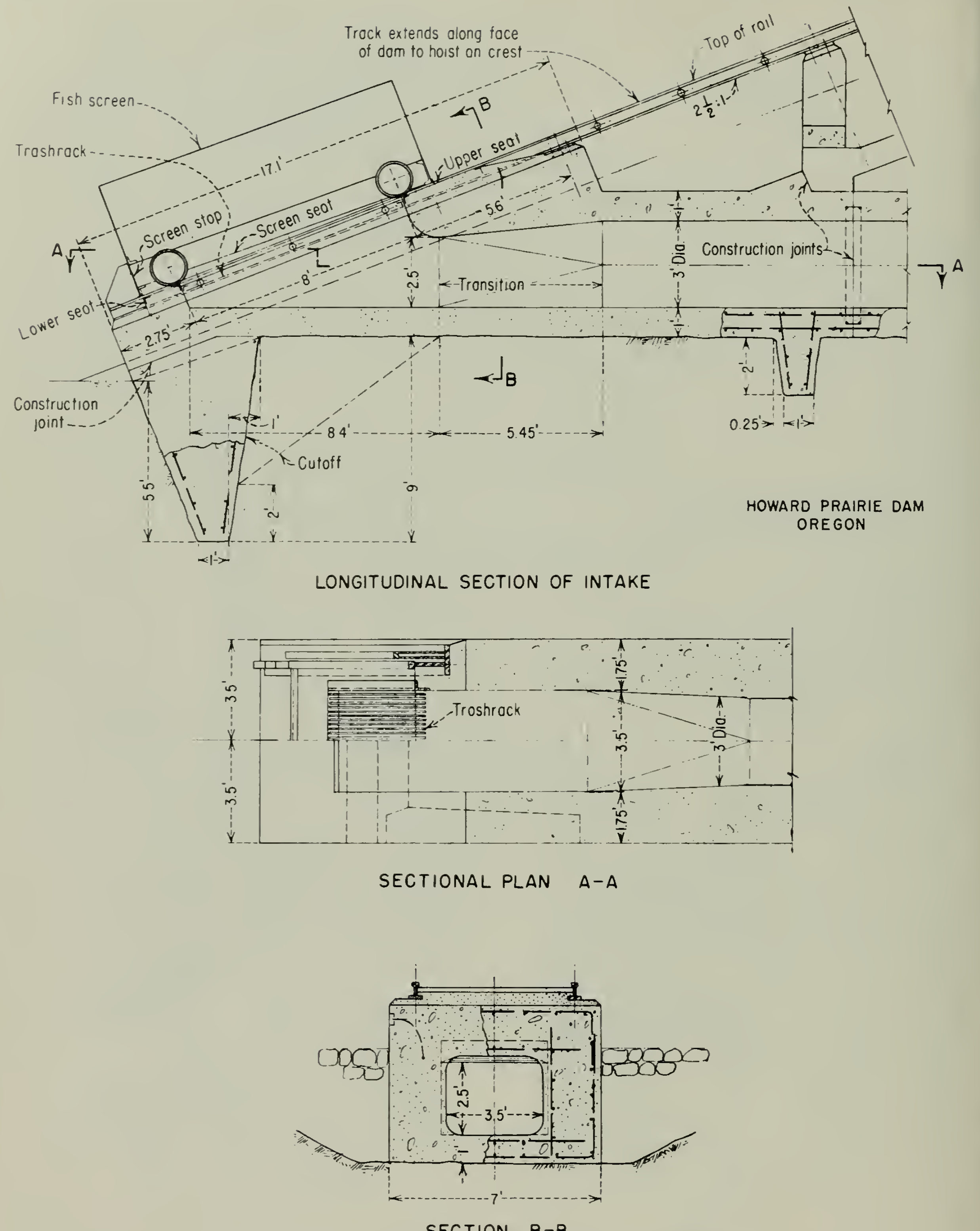


Figure 263.—Typical intake with sloping entrance.

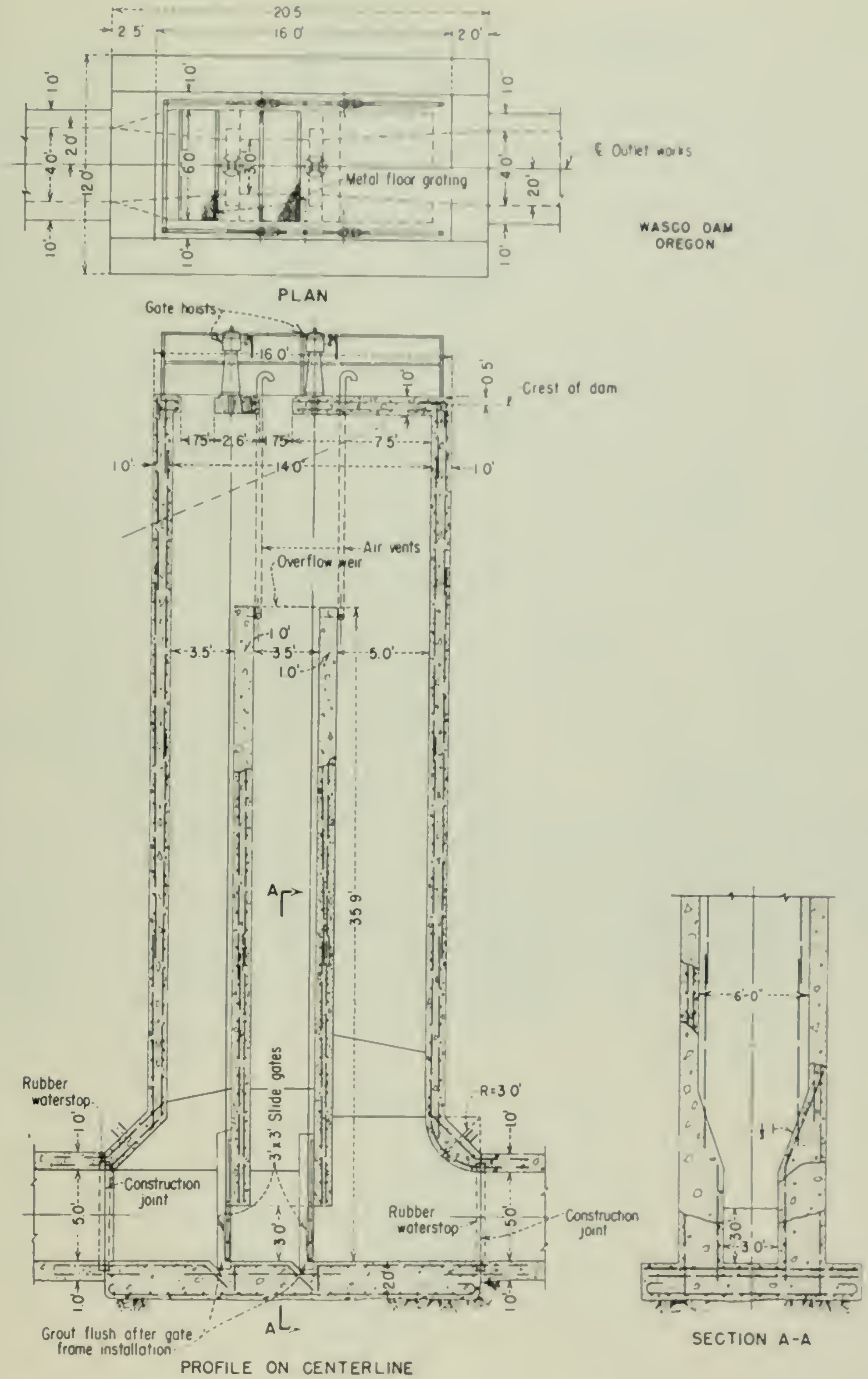


Figure 264. Typical shaft for slide gate control.

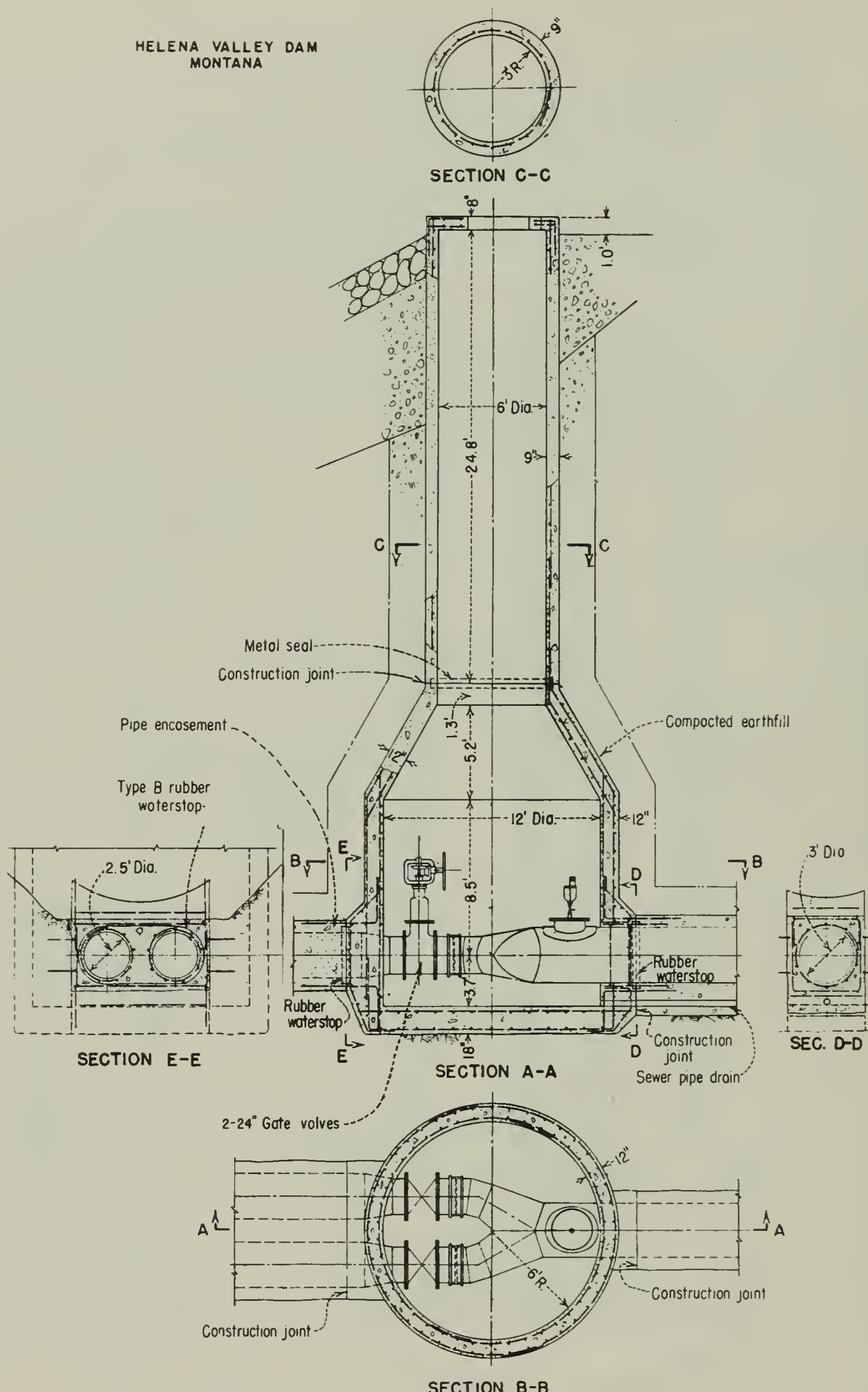


Figure 265. **Dry well shaft for gate-valve control installation.**

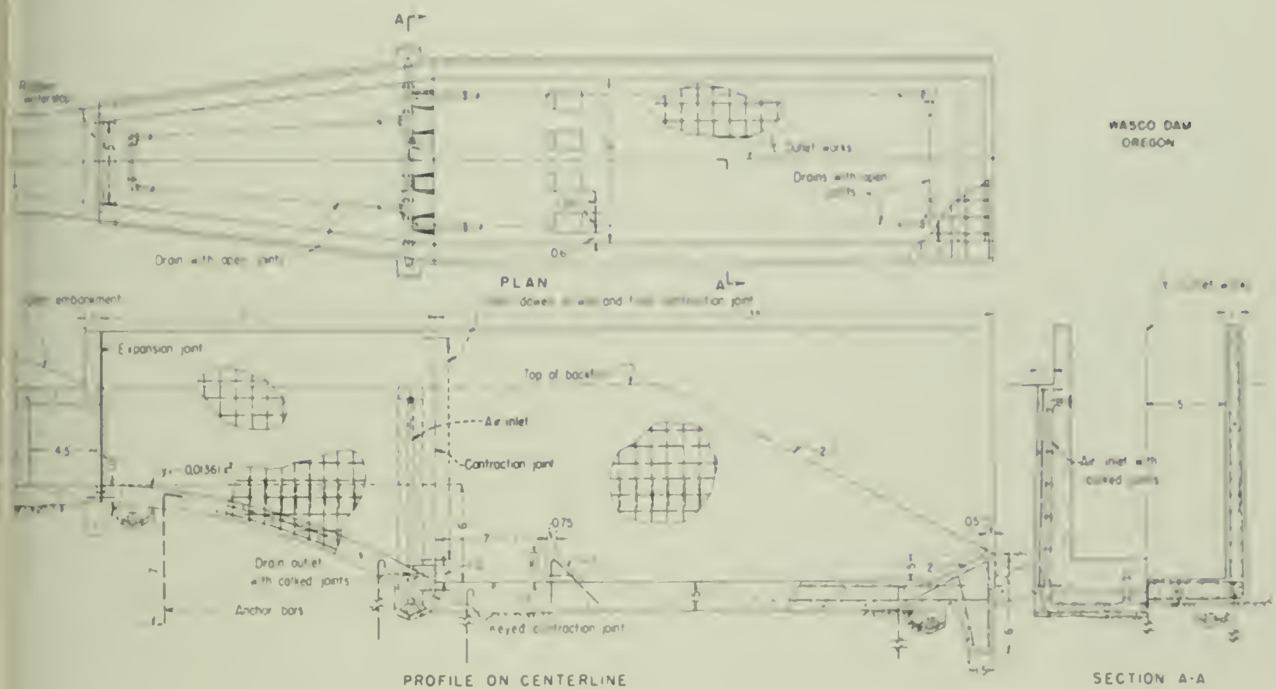


Figure 266. Typical hydraulic jump stilling basin.

Diversion During Construction

E. R. LEWANDOWSKI [1]

A. DIVERSION REQUIREMENTS

238. *General.*—The design for a dam which is to be constructed across a stream channel must consider diversion of the streamflow around or through the dam site during the construction period. The extent of the diversion problem will vary with the size and flood potential of the stream; at some dam sites diversion may be costly and time-consuming and may affect the scheduling of construction activities, while at other sites it may not offer any great difficulties. However, a diversion problem exists to some extent at all sites except those located offstream, and the selection of the most appropriate scheme for handling the flow of the stream during construction is important to secure economy in the cost of the dam. The scheme selected ordinarily will represent a compromise between the cost of the diversion facilities and the amount of risk involved. The proper diversion plan will minimize serious potential flood damage to the work in progress at a minimum of expense. The following factors should be considered in a study to determine the best diversion scheme:

(1) Characteristics of streamflow.
(2) Size and frequency of diversion flood.
(3) Methods of diversion
(4) Specifications requirements.

239. *Characteristics of Streamflow.*—Streamflow records provide the most reliable information regarding streamflow characteristics, and should be consulted whenever available.

Depending upon the size of the drainage area and its geographical location, floods on a stream may be the result of snowmelt, seasonal rains, or cloudbursts. Because each of these types of runoff have their peak flows and their periods of low flow at different times of the year, the nature of runoff will influence the selection of the diversion scheme. A site subject only to snowmelt floods will not have to be provided with elaborate measures for use later in the construction season. A site where seasonal rains may occur will require only the minimum of diversion provisions for the rest of the year. A stream subject to cloudbursts which may occur at any time is the most unpredictable and probably will require the most elaborate diversion scheme, since the contractor must be prepared to handle both the low flows and floodflows at all times during the construction period.

240. *Selection of Diversion Flood.*—Usually, it is not economically feasible to plan on diverting the largest flood that has ever occurred or may be expected to occur at the site, and consequently some lesser requirement must be decided upon. This, therefore, brings up the question as to how much risk is involved in the diversion scheme under consideration. In the case of an earthfill dam, where considerable areas of foundation and structure excavation are exposed, or where overtopping of the embankment while under construction may result in serious damage or loss of the partially completed work, the importance of eliminating the risk of flooding is relatively great. This consideration is not as important in the case of a concrete dam since the floodwaters may, if the location of appurtenant structures permits, overtop the dam with little or no adverse effect.

In selecting the flood to be used in the diversion designs, consideration should be given to the following.

(1) How long the work will be under construction, to determine the number of flood seasons which will be encountered.

[1] Engineer, Earth Dams Section, Bureau of Reclamation

(2) The cost of possible damage to work completed or still under construction if it is flooded.

(3) The cost of delay to completion of the work, including the cost of forcing the contractor's equipment to remain idle while the flood damage is being repaired.

(4) The safety of workmen and possibly the safety of downstream inhabitants in case the failure of diversion works results in unnatural flooding.

After an analysis of these factors is made, the cost of increasing the protective works to handle progressively larger floods can be compared to the cost of damages resulting if such floods occurred without the increased protective work. Judgment can then be used in determining the amount of risk that is warranted.

For small dams which will be constructed in a single season, only the floods which may occur during that season need be considered. For most small dams, involving at the most two construction seasons, it should be sufficiently conservative to provide for the largest flood likely to occur in a 5-year period. However, to minimize the risk, the diversion capacity might be increased to handle the 10-year or larger flood if it can be done at little additional cost. The methods for determining floods of 5-year, 10-year or less frequency are discussed in section 42.

It should be considered that floods may be recurrent; therefore, if the diversion scheme involves temporary storage of cloudburst-type runoff, facilities must be provided to evacuate such storage within a reasonable period of time, usually a few days.

B. METHODS OF DIVERSION

241. ***General.***—The method or scheme of diverting floods during construction depends on the magnitude of the flood to be diverted; the physical characteristics of the site; the type of dam to be constructed; the nature of the appurtenant works, such as the spillway, penstocks, or outlet works; and the probable sequence of construction operations. The objective is to select the optimum scheme considering practicability, cost, and the risks involved. The diversion works should be such that they may be incorporated into the overall construction program with a minimum of loss, damage, or delay.

Common practice for diverting streams during construction utilizes one or a combination of the following provisions: Tunnels driven through the abutments, conduits through or under the dam, temporary channels through the dam, or multiple-stage diversion over the tops of alternate construction blocks of a concrete dam. Outlet works conduits or tunnels frequently are made sufficiently large to carry the diversion flow. On a small stream the flow may be bypassed by the installation of a temporary wood or metal flume or pipeline, or the flow may be impounded behind the dam during its construction, pumps being used if necessary to control the water surface. Figures 267 and 268 show flumes used to divert the streamflow during the construction of an earthfill dam and a concrete dam, respectively. In any case, barriers are constructed across or along the stream channel in order that the site, or portions thereof, may be unwatered and construction can proceed without interruption.

A common problem is the meeting of downstream requirements when the entire flow of the stream is stopped during closure of the diversion works. Downstream requirements may demand that a small flow be maintained at all times, in which case the contractor must provide the required flow by pumping or by other means (bypasses or siphons) until water is stored in the reservoir to a sufficient level so that it may be released by gravity flow through the outlet works.

Figure 269 shows how diversion of the river was accomplished during the construction of Folsom Dam and Powerplant on the American River in California. Although this structure is considerably larger than the dams discussed in this text, this photograph is included because it illustrates many of the diversion principles discussed in this chapter. The river, flowing from top to bottom in the picture, is being diverted through a tunnel; "a" and "b" mark the inlet and outlet portals, respectively. Construction is proceeding in the original river channel between earthfill cofferdams

Figure 267. **Temporary diversion flume through an earthfill dam site. (Willow Creek Dam, CBT 245-704-3287.)**

"c" and "d." Discharge from pipe "e" at the lower left in the photograph is from unwatering of the foundation. Since it was impracticable to provide sufficient diversion tunnel capacity to handle the large anticipated spring floods, the contractor made provisions to minimize damage that would result from overtopping of the cofferdam. These provisions included the following:

(1) Placing concrete in alternate low blocks in the dam "f" to permit overflowing with a minimum of damage;

(2) Construction of an auxiliary rockfill and cellular steel sheet-piling cofferdam "g" to protect the powerplant excavation "h" from being flooded by overtopping of the cofferdam; and

(3) Early construction of the permanent training wall "i" to take advantage of the protection it affords.

242. ***Tunnels.***—It is usually not feasible to do a significant amount of foundation work in a narrow canyon until the stream is diverted. In this situation a tunnel may prove the most feasible means for diversion, either for a concrete dam or for an earthfill dam The streamflow is bypassed around the construction area through tunnels in one or both abutments. If tunnel spillways or tunnel outlet works are provided in the design, it usually proves economical to utilize them in the diversion plan If the upstream portion of the permanent tunnel is above the streambed elevation, a temporary upstream diversion adit can be provided to effect a stream-level bypass. Figure 270 shows such an adit which was constructed at Seminoe Dam to permit diversion through the spillway tunnel. The diversion adit leads from the streambed to the intersection of the horizontal

portion of the spillway tunnel and the inclined shaft leading from the spillway gate structure. First stages of construction of the spillway gate structure can be seen in the upper right-hand portion of the photograph.

The advisability of lining the diversion tunnel will be influenced by the cost of a lined tunnel compared with that of a larger unlined tunnel of equal carrying capacity; the nature of the rock in the tunnel, as to whether it can stand unsupported and unprotected during the passage of the diversion flows; and the permeability of the material through which the tunnel is carried, as it will affect the amount of leakage through or around the abutment.

Some means of shutting off diversion flows must be provided; in addition, some means of regulating the flow through the diversion tunnel may be necessary. Closure devices may consist of a timber, concrete, or steel bulkhead gate; a slide gate; or stoplogs. Regulation of flow to satisfy downstream needs after storage of water in the reservoir has been started can be accomplished by the use of a slide gate, wheel-type regulating gate, or temporary bypass until the water surface in the reservoir reaches the level of the outlet works intake. Figure 271 shows temporary closure of the upstream portal of a diversion tunnel by means of a wooden bulkhead. The pipes in the bulkhead were provided to pass flows to meet downstream requirements until the reservoir level rose to the lip of the outlet works intake, when the pipes were closed by the flap gates shown. The pipes extended through the diversion tunnel plug and ultimately they were grouted to complete the closure.

Permanent closure of the diversion tunnel is made by placing a concrete plug in the tunnel. Where a temporary diversion adit joins a permanent tunnel, the plug is usually placed immedi-

Figure 268. **Temporary diversion flume used during construction of a concrete dam. (Horsetooth Feeder Canal Tunnel No. 1, CBT 245-704-330.)**

ately upstream from the intersection as shown on figure 272. Keyways may be excavated into the rock or formed into the lining to insure adequate shear resistance between the plug and the rock or lining. After the plug has been placed and the concrete cooled, grout is forced through previously installed grout connections into the contact between the plug and the surrounding rock or concrete lining to insure a watertight joint. Sample specifications for this grouting are contained in appendix G.

243. *Conduits.*—The outlet works for an earthfill dam often entails the construction of a conduit which may be utilized for diversion during construction of the dam. This method for handling the diversion flows is an economical one, especially if the size of the conduit required for the outlet works is adequate to carry the diversion flows. Where diversion requirements exceed the capacity of the completed outlet works, an increase in capacity can be obtained by delaying the installation of gates, valves, pipe, and trashracks until the need for diversion is over. Increased capacity also can be obtained by increasing the height of the cofferdam, thereby increasing the head. In some instances the storage capacity of the reservoir at lower elevations may be such that much of the diversion design flood may be temporarily retained and then evacuated through a diversion conduit of smaller capacity than would be required to discharge the peak of the flood.

In cases where the intake to the outlet works conduit is above the level of the streambed, an auxiliary stream-level conduit may be provided to join the lower portion of the permanent conduit. Such an auxiliary conduit is shown in figure 273. Permanent closure of this auxiliary conduit after diversion is completed can be accomplished in the same manner as that outlined in section 242. A concrete plug in an auxiliary diversion conduit is shown in figure 274.

Diversion conduits at stream level are sometimes provided through a concrete dam. These openings are provided with keyways, metal seals,

Figure 269. **Diversion of the river during construction of Folsom Dam and Powerplant (Corps of Engineers and Bureau of Reclamation). (AR–1627–CV.)**

Figure 270. **Diversion adit and upstream cofferdam at Seminoe Dam. (891, Kendrick.)**

and grouting systems and must be permanently closed throughout their entire length in the manner prescribed for placing tunnel plugs.

244. *Temporary Diversion Channel—Earthfill Dams.*—At sites where it may not be economical to provide a tunnel or conduit large enough to pass the diversion design flood, a temporary channel involving a gap through the earthfill dam may be utilized to divert streamflows while the remainder of the embankment is being constructed, as shown in figure 275. Though the dam shown is larger than the size of structures considered in this text, the methods and requirements for diversion through a temporary gap in the dam are the same. This method is adaptable to wide sites; obviously it cannot be used in narrow canyons. However, it is in the wider valleys where the diversion flows are likely to be too large to be economically carried in tunnels or conduits.

Before the stream is diverted, the foundation preparation required for the dam should be completed in the area where the temporary opening will be left through the embankment. This preparation would include excavation and refilling of a cutoff trench, if one is to be constructed. The stream is then channeled through this area, after which the foundation work in the remainder of the streambed is completed. The portion of the embankment to either side of the diversion opening may then be completed. The side slopes of the opening should not be steeper than 4 to 1 to facilitate filling of the gap at the end of the construction period and to decrease the danger of cracking of the embankment due to differential settlement. The flat slope also provides a good bonding surface between the previously constructed embankment and the material to be placed.

The bottom grade of the temporary channel through the embankment should be the same as the original streambed so that erosion in the channel will be minimized. The width of opening will depend on the magnitude of the diversion

Figure 271. **Bulkhead closure of diversion tunnel with pipes to pass flows for downstream requirements. (Granby Dam, 400–GL–216.)**

design flood and on considerations of the equipment capabilities for filling the gap in the time which will be available.

The diversion is carried through the opening in the dam until sufficient progress is made in the completion of the embankment and appurtenant works so that floods can be carried safely through the completed or partially completed spillway. Closure of the gap in the embankment can then be made. To reduce the risk of the rising water surface in the reservoir overtopping the embankment being placed in the closure section, a period should be selected during the construction season when large floods are least likely to occur. Construction equipment should be mobilized so that the gap can be filled as quickly as possible to an elevation which will permit discharge of a flood, should one occur, through the spillway. The average rate of embankment placement must be such that the gap can be filled faster than the water rises in the reservoir. The capability of the contractor's plant to meet this requirement may be gaged by considering the average rate of embankment placing he will have to attain in order to complete the entire dam within the contract period, taking into account the number of days during the contract period that the weather will likely be suitable for embankment construction.

Care must be exercised during the filling of the gap so that the quality of work is not sacrificed due to the exigency of the situation. This is of great importance because frequently the diversion gap is in the area where the dam will be of maximum height. Extreme care must be used in order to obtain required densities and thus avoid excessive settlement of the completed embankment.

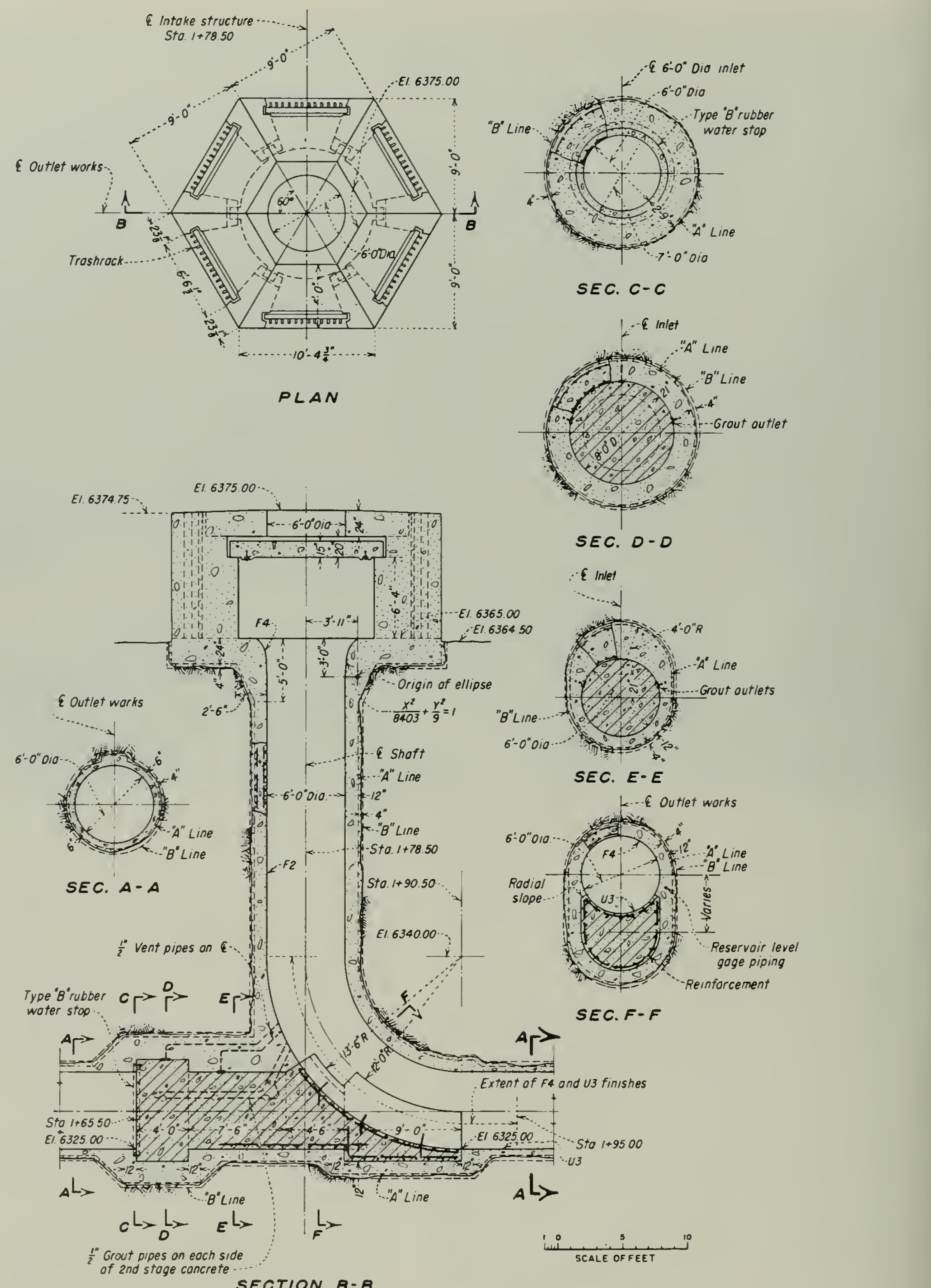


Figure 272. Concrete plug in diversion adit. From drawing No. 339–D–199.

Figure 273. **Diversion through auxiliary stream-level conduit. Construction is in progress on outlet works intake. (Heart Butte Dam, DR-P-319.)**

Special attention must also be given to bonding of the newly placed material with the earthfill previously placed.

245. ***Multiple-Stage Diversion for Concrete Dams.***—The multiple-stage method of diversion over the tops of alternate low construction blocks or through diversion conduits in a concrete dam requires shifting of the cofferdam from one side of the river to the other during construction. During the first stage, the flow is restricted to one portion of the stream channel while the dam is constructed to a safe elevation in the remainder of the channel. In the second stage, the cofferdam is shifted and the stream is carried over low blocks or through diversion conduits in the constructed section of the dam while work proceeds on the unconstructed portion of the dam. The dam is then carried to its ultimate height, with diversion ultimately being made through the spillway, penstock, or permanent outlets. Figure 276 shows diversion through a conduit in a concrete dam, with excess flow over the low blocks.

246. ***Cofferdams.*** A cofferdam is a temporary dam or barrier used to divert the stream or to enclose an area during construction. The design of an adequate cofferdam involves the problem of construction economics. Where the construction is timed so that the foundation work can be executed during the low water season, use of cofferdams can be held to a minimum. Where the streamflow characteristics are such that this is not practicable, the cofferdam must be so designed

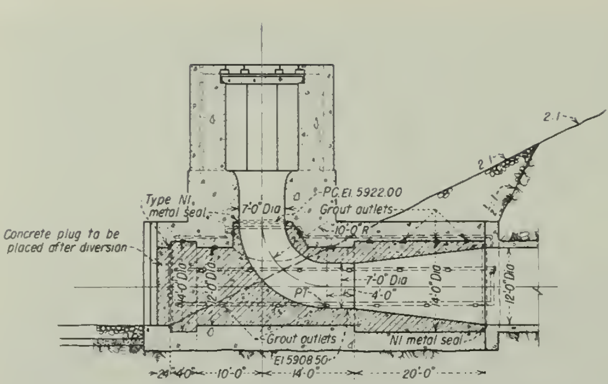


Figure 274. Detail of a concrete plug in an auxiliary diversion conduit. From drawing 526–D–146.

that it is not only safe, but also of the optimum height. The height to which a cofferdam should be constructed may involve an economic study of cofferdam height versus diversion works capacity, including routing studies of the diversion design flood, especially when the outlet works requirements are small. If outlet works requirements dictate a relatively large outlet conduit or tunnel, diversion flows ordinarily may be accommodated without a high cofferdam. It should be remembered that floodwater accumulated behind the cofferdam must be evacuated in time to accommodate a recurrent storm. The maximum height to which it is feasible to construct the cofferdam without encroaching upon the area to be occupied by the dam must also be considered. Furthermore, the design of the cofferdam must take into consideration the effect that excavation and unwatering of the foundation of the dam will have on its stability, and must anticipate removal, salvage, and other factors.

Generally, cofferdams are constructed of materials available at the site. The two types

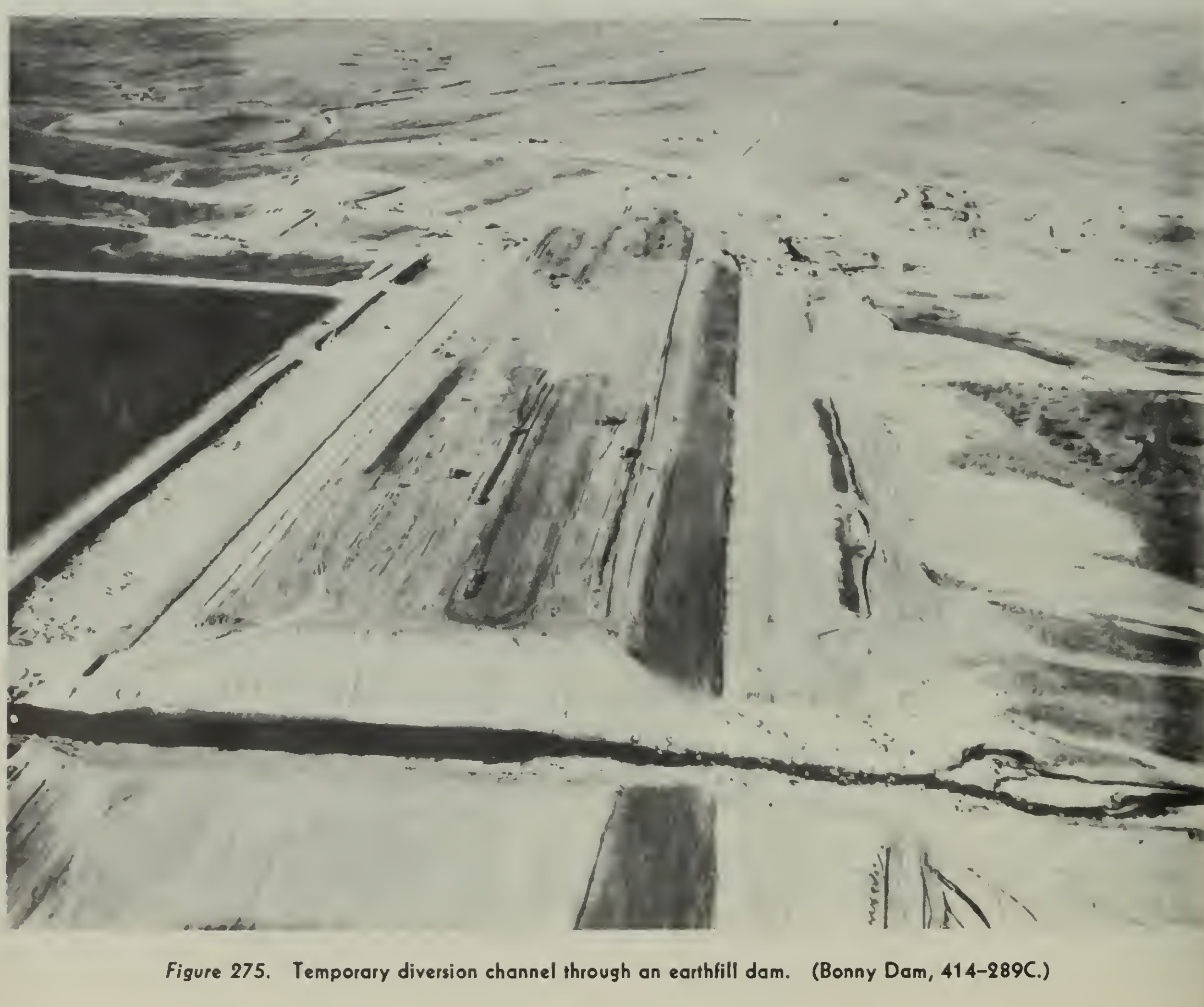


Figure 275. Temporary diversion channel through an earthfill dam. (Bonny Dam, 414–289C.)

Figure 276. **Flows through diversion opening and over low blocks of a concrete dam. (Olympus Dam, EPA–PS–330–CBT.)**

normally used in the construction of dams are earthfill cofferdams and rockfill cofferdams, the design considerations of which closely follow those for permanent small dams of the same type. Figure 270 shows the construction of an earth and rockfill cofferdam. Other types not as common include timber or concrete cribs filled with earth or rock, and cellular steel cofferdams filled with earth or rock. Figure 277 shows a combination of several types. In this case, the major portion of the cofferdam consists of an earth and rock embankment, and steel sheet piling was used to effect final closure in swift water. Figure 269 shows the use of both earthfill cofferdams and cofferdams formed of steel-piling cells. Cellular steel cofferdams and steel sheet piling are adaptable to confined areas where currents are swift.

If the cofferdam can be designed so that it is permanent and adds to the structural stability of the dam, it will have a decided economic advantage. In some earthfill dams the cofferdam can be incorporated into the main embankment. In such instances, the saving is twofold—the amount saved by reducing the amount of embankment material required, and the amount saved by not having to remove the cofferdam when no longer needed.

C. SPECIFICATIONS REQUIREMENTS

247. *Contractor's Responsibilities.*—It is general practice to require the contractor to assume responsibility for the diversion of the stream during the construction of the dam and appurtenant structures. The requirement should be defined by appropriate paragraphs in the specifications which describe the contractor's responsibilities and inform him as to what provisions, if

484556 O—60——28

Figure 277. **Upstream cofferdam of steel sheet piling and earthfill diverting streamflow into tunnel. (Green Mountain Dam, GM–283–CBT.)**

any, have been incorporated in the design to facilitate construction. Usually the specifications should not prescribe the capacity of the diversion works, nor the details of the diversion method to be used, but hydrographs prepared from streamflow records, if available, should be included. Also, the specifications usually require that the contractor's diversion plan be subject to the owner's approval.

In some cases, such as in constructing a concrete gravity dam in a wide canyon, the entire diversion scheme might be left in the contractor's hands, with the expectation that the flexibility afforded to the contractor's operations by allowing him to choose the scheme of diversion will be reflected in low bids. Since various contractors will usually present different schemes, the schedule of bids in such instances should require diversion of the river to be included as a lump-sum bid. Sometimes a few pertinent paragraphs are appropriate in the specifications giving stipulations which affect the contractor's construction procedures. For example, for an earthfill dam where diversion by means of a temporary channel is feasible or contemplated, the specifications may permit the contractor to divert the stream over the embankment placed in the completed cutoff trench, but usually would prohibit him from making final closure of the diversion works until the dam has been constructed to an elevation well above the spillway crest. Also, the contractor may be required to have the concrete in the spillway and outlet works essentially completed prior to closure of the temporary channel.

These, or similar restrictions, tend to guide the contractor toward a safe diversion plan. However, to further define the contractor's responsibility, other statements should be made to the effect that the contractor shall be responsible for and shall repair at his expense any damage

to the foundation, structures, or any other part of the work caused by flood, water, or failure of any part of the diversion or protective works. The contractor should also be cautioned concerning the use of the hydrographs by a statement to the effect that the contracting authority does not guarantee the reliability or accuracy of any of the hydrographs and assumes no responsibility for any deductions, conclusions, or interpretations that may be made from them.

Sample specifications regarding diversion during construction are included in appendix G.

248. ***Designer's Responsibilities.*** For difficult diversion situations, it may prove economical for the owner to assume the responsibility for the diversion plan. One reason for this is that contractors tend to increase bid prices for diversion of the stream if the specifications contain many restrictions and there is a large amount of risk involved. Where a dam is to be constructed in a narrow gorge, a definite scheme of cofferdams and tunnels might be specified, since here the loss of life and property damage might be heavy if a cofferdam built at the contractor's risk were to fail.

Another point to consider is that many times the orderly sequence of constructing various stages of the entire project depends on a particular diversion scheme being used. If the responsibility for diversion rests on the contractor, he may pursue a different diversion scheme, with possible delay to completion of the entire project. This could result in a delay in delivery of irrigation water or in generation of power, or both, with a subsequent loss in revenue.

If the owner assumes responsibility for the diversion scheme, it is important that the diversion scheme be realistic in all respects, and compatible with the probable ability and capacity of the contractor's construction plant.

D. BIBLIOGRAPHY

249. ***Bibliography.***

Ackerman, A. J., and Locher, C. H., "Construction Planning and Plant," McGraw-Hill Book Co., Inc., New York, 1940.

Maintenance and Operation

H. G. ARTHUR [1]

A. GENERAL

250. *Inspections and Schedule of Observations.*—Arrangements should be made, immediately following the completion of a dam, for periodic inspection of the structure and all of the operating equipment. Adequate measures to accomplish this are usually taken on the more important structures but frequently are neglected on small dams. General responsibility for such structures may lie with a State, county, municipality, or a special board or commission endowed with administrative powers. The owner should advise such authority of the definite arrangements being made for periodic inspection and report by a responsible person who is informed of the hazards. For more important structures, the inspection should be conducted by an engineer. In remote locations, arrangements may be made with a forester, a minor county official, or a nearby rancher for inspection of small structures.

[1] Engineer, Earth Dams Section, Bureau of Reclamation.

251. *Maintenance and Operating Instructions.*—Written instructions for maintenance and operation of the structures and equipment should be prepared as part of the design function and furnished to the owner or operating agency. These instructions should establish the frequency of and describe the extent and nature of inspections.

The instructions should also provide for the routine servicing of mechanical equipment where gates and valves are provided, and should include such instructions furnished by the manufacturer. The inspections and servicing should be accomplished in accordance with the principles discussed in part B of this chapter.

The instructions should also include detailed discussions of the proper operation of gates and valves from both mechanical and functional viewpoints. If a spillway is controlled by manually operated gates, specific instructions should be given regarding the operation of the gates during flood inflows into the reservoir.

B. INSPECTION AND MAINTENANCE OF DAMS

252. *Earthfill Dams.*—(a) *General Information.*—Routine maintenance of embankment slopes and the crests of earth embankments can be expected. However, any unusual conditions which may adversely affect the safety of an earthfill dam should be reported promptly. Any abnormal requirements for maintenance should also be reported.

(b) *Inspection of Embankments and Foundations.*—The embankment, abutments, and visible portions of the foundation adjacent to an earth embankment should be inspected at regular intervals for evidence of development of unfavorable conditions.

During rapid filling of the reservoir, the downstream slope of the embankment and the foundation downstream from the embankment should be carefully inspected at frequent intervals for indications of cracks, slides, sloughs, subsidences, impairment of slope protection, springs, seeps, or boggy areas caused by seepage from the reservoir. The upstream slope of the embankment should also be carefully inspected after periods of

sustained high-velocity winds, and as the reservoir water surface is being drawn down, for evidence of cracks, slides, sloughs, subsidences, or damages to the slope protection, such as displacement of riprap or other signs of serious erosion.

During periods of low reservoir level, the exposed portions of the abutments and the reservoir floor should be carefully examined for sinks or seepage holes, unusual beaching conditions, or cracking. During periods of sustained high reservoir level, monthly inspections should be made of the embankment, with particular attention being given to the crest of the dam, to the visible portions of the upstream slope protection, to the downstream slopes, and to areas downstream from the dam for evidence of abnormal development. The frequency of inspections may be decreased after several seasons of operation have disclosed no abnormal conditions.

(c) *Reporting of Abnormal Developments.*—The occurrence of unusual conditions should be reported immediately to the operating agency or owner of the dam by letter, telegram, or telephone, depending upon the nature of the development and the apparent urgency for repair. The description of slides, sloughs, or sudden subsidences should include the location, extent, rate of subsidence, effects of adjoining structures, reservoir and tailwater elevations, prevailing weather conditions, and other facts believed to be pertinent.

Information regarding the development of springs, seeps, and boggy areas should include such data as the location and size of the affected areas, the estimated discharge, the nature of discharge (whether clear or cloudy water), and the reservoir tailwater elevations. To facilitate analysis of conditions, a map should be prepared showing the extent of all seeped areas, springs, and data such as dates and reservoir levels at the time of observation. This map should be revised periodically.

253. *Concrete Dams and Concrete Structures.*—(a) *General Information.*—All concrete structures, including dams, should be periodically inspected, with the following objectives in mind:

(1) To verify the safety of structures and facilities.

(2) To disclose conditions which might cause disruption or failure of operation.

(3) To determine the adequacy of structures and facilities to serve the purpose for which they were designed and are being used.

(4) To note the extent of deterioration as a basis for planning maintenance, repair, and rehabilitation.

(b) *Inspection of Concrete Structures.*—Annual inspections should be made by operating personnel and, at intervals of not greater than 6 years, major concrete structures should be inspected by a board of qualified engineers. Annual inspections should be made of all portions of the structures readily accessible and all other portions where there is reason to believe that damage may have occurred. Inspections by the board of engineers should be more thorough and detailed than those made by operating forces and should include portions of the structures not ordinarily accessible, such as penstocks, conduits, etc. These inspections should be scheduled during periods of low water to check the condition of structures normally submerged and during periods of maximum water level to check structural behavior under full load, or during maximum spillway discharges. The inspections should cover such items as:

(1) Abnormal settlements, heaving, deflections, or lateral movement of concrete structures.

(2) Cracking or spalling of concrete and opening of contraction joints.

(3) Deterioration, erosion, or cavitation of concrete.

(4) Abnormal leakage through foundation or formed drains or through concrete surfaces, construction joints, or contraction joints.

(5) Possible undermining of the downstream toe or other foundation damage.

(6) Unusual or inadequate operational behavior.

A written report of this inspection should be made to the owner or operating agency discussing the structures and conditions observed and recommendations for corrective measures where required.

(c) *Inspection of Channels and Surrounding Areas.*—Channels and surrounding areas, including backfill adjacent to concrete structures and embankment not included in the limits of an earthfill dam, should be inspected in conjunction with the concrete structures. These inspections should cover such items as:

(1) Channel bank or bed erosion and silting.

(2) Condition of riprap areas.

(3) Presence and condition of undergrowth in bottoms and on sides of channels and estimated effect on tailwater levels.

(4) River aggradation or degradation and possible effect on hydraulic operation of structures involved.

(5) Abnormal subsidence of backfill or embankment areas.

(6) Unusual or inadequate operational behavior.

254. ***Mechanical Equipment.***—Periodic inspection of spillway gates and tests of operating equipment should be made by an engineer or a mechanic familiar with the purposes of the equipment. Inlet and outlet gates and valves should be tested regularly to see that they work freely. All gates and valves should be exercised at least annually, to determine that they are in good operating contion. Mechanical equipment should be lubricated and serviced in accordance with the manufacturer's instructions.

Trashracks should be cleaned of debris and accumulated sediment; metalwork should be painted to prevent rusting.

C. OPERATION

255. ***Storage Dams.***—The dams discussed in this text normally will be used to store water for supplementary irrigation, domestic water supply, recreational purposes, stock ponds, or auxiliary flood control in tributaries of main streams. Their operation will rarely require continuous attention, except at seasonal intervals. If warranted, there should be an operator's house with telephone service, at or near the control works of dams. Also, the operator should have available a supply of small tools, sandbags, and other maintenance and emergency equipment.

Besides control for purposes of distribution, the water level in the reservoir may require a change at regular intervals to prevent propagation of malarial or pest mosquitoes, and to evade algae or other aquatic growth. The pond level on storage dams may also have to be drawn down upon receipt of storm warnings to provide storage for floodwaters.

The stimulation and protection of growth of vegetative cover to retard erosion on the slopes of the reservoir, on the borrow pits used in construction, and on the slopes of earthfill dams not otherwise protected is an important item of maintenance to which careful attention should be given. This cover is an essential item of protection against erosion and sloughing of banks, as well as of beautification of the structure, and may have an important influence on the cost of repairs.

Expert advice on suppression of algae growth in reservoirs should be obtained and followed, and no chemicals should be introduced into a reservoir without competent advice.

Instructions for operation of mechanical equipment should be followed closely so as to prevent damage to any of the installations through improper operation. Instructions given for the manual control of spillway gates during the occurrence of floodflows into the reservoir should be followed in detail, and operating personnel should not presume to interfere with the operation of the spillway in the manner which was contemplated in the design as explained in the maintenance and operating instructions.

256. ***Diversion Dams.***—Diversion dams are dams built for the purpose of raising the level of the stream and not for purposes of storage or equalization of flow. Such dams may divert flows into canals for irrigation of lowlands in the stream valleys or to spreading grounds for replenishment of ground-water storage.

Diversion dams are almost invariably overflow dams or have long overflow sections. Control gates are usually supplied so that the required diversion level may be maintained in spite of fluctuations in streamflow or in order to pass portions of the flow as needed to satisfy downstream water rights. This mechanical equipment should be operated and maintained in accordance with the instructions furnished as a part of the design function.

Diversion dams are often founded on sandy or gravelly streambed materials. In such cases, their stability may be insured by a broad base with cutoff walls. Such dams must be safeguarded by frequent inspections for evidence of piping or boils below the dams or for an increase in volume

of seepage appearing at the downstream toe. The downstream apron is usually protected at the toe by heavy riprap. After floods, the streambed should be examined and the riprap renewed and repaired if necessary.

257. *Flood Detention Reservoirs.*—Flood detention reservoirs serve to reduce flood peaks by the temporary storage of that part of the flow which is in excess of the capacity of the spillway or outlet works of the dam. All reservoirs or pools produce some detention effect.

Structures built for the specific purpose of flood control by detention may be built with outlets which will automatically control the rate of release within safe limits. Overflow spillways are also provided in order to protect the dams even at the expense of possible flood damage below the structure, should a flood occur larger than that for which the dam was designed to control.

In addition to general inspections, the outlet works of such structures should be kept clear of soil deposits and debris which might affect their proper functioning.

258. *Changes in Operating Plan.*—A dam built for purposes of flood control may be diverted from its intended use by reason of the demand of a community for full storage for irrigation or water supply. Such demands, if acceded to, may result in a dangerous situation and possibly in the complete loss of the dam by overflow in the event of excessive runoff. A change in the operation of a structure or dam should not be made without a complete appraisal of the effects which such a change will have upon the structural and functional design of the structure.

Raising the height of a dam is sometimes undertaken without due consideration of the relation of the increased pressures to the limitations of the original design. No structural changes should be made without reference to the original plans or without the advice of an experienced engineer, preferably the designer.

The capacity of a storage reservoir should not be increased by placing stoplogs or other obstructions in an open-crest spillway without reference to the original plans and contemplated method of operation or without the advice of an experienced engineer. Such devices may operate in such a manner as to effectively reduce the ability of the reservoir and dam to safely store and pass the predicted inflow design flood.

259. *Coordination of Multiple Uses.*—Storage dams may be operated for more than one purpose. Multiple use may be made of the same storage space, or various head ranges in the same reservoir may be utilized for different purposes, such as flood control, power, irrigation, recreation, water supply, or navigation. Such combined operation requires very careful planning and control, as some of the uses are not compatible with others. For example, a power user, in order to be sure of the maximum amount of firm power, may wish to have the storage full when flood hazard is imminent and the need for available reservoir capacity greatest. Such combined operation is possible only with loss of some measure of benefit to one or all of the participants. In spite of these difficulties, conditions often make it desirable to permit such multiple use. Where such allocations are known in advance, they may have an influence on the design of the control devices. Careful management is important in the operation of multiple-use reservoirs in order to maintain a balanced perspective in the matter of relative values.

The method of operation of a multiple-use reservoir should not be arbitrarily changed without consideration of the effect of such a change upon the operation of the dam and upon the maximum water surface which will result, should the design inflow flood occur.

Estimating Rainfall Runoff from Soil and Cover Data

A-1. ***Introduction.***—Although an engineer attempting to estimate the amount of runoff that will result from a given amount of precipitation on a specific area must exercise considerable judgment, the more information he has available the more accurate will be his estimate. The information presented in this appendix provides a means whereby an engineer may obtain an estimate of direct runoff from a given amount of rainfall. It is taken from the "Hydrology Guide for Use in Watershed Planning," [1] published by the Soil Conservation Service, with some editorial changes to meet the purpose of this text.

A-2. ***Hydrologic Soil Groups.***[2]—(a) *Purpose.*—Watershed soil determinations are used in the preparation *of hydrologic soil-cover complexes* (sec. A-4), which in turn are used in estimating direct runoff. Four major soil groups are used. The soils are classified on the basis of intake of water at the end of long-duration storms occurring after prior wetting and opportunity for swelling, and without the protective effects of vegetation.

The major hydrologic soil groups are:

A. (Lowest runoff potential.) Includes deep sands with very little silt and clay, also deep, rapidly permeable loess.

B. Mostly sandy soils less deep than group A, and loess less deep or less aggregated than group A, but the group as a whole has above-average infiltration after thorough wetting.

C. Comprises shallow soils and soils containing considerable clay and colloid, though less than those of group D. The group has below-average infiltration after presaturation.

D. (Highest runoff potential.) The group includes mostly clays of high swelling percent, but it also includes some shallow soils with nearly impermeable subhorizons near the surface.

The classification of soils into the major hydrologic soil groups may be done on the basis of the soil array, as discussed in the next section, provided agricultural soil maps are available. If such maps are not available, the soils will have to be classified on the basis of judgment.

(b) *Determination of Hydrologic Soil Groups.*—Each watershed is classified as being in one of the four major hydrologic groups given above. The soil array, table A-1, is used to find the average classification of soils. For example, a given watershed has 80 percent of its area in the B soil group and 20 percent in the C. The C soils are interspersed with the B soils. The watershed is classed as a B soil group area. The C soil group cannot be handled separately unless it is a compact area, in which case the watershed is divided into two parts which are handled as individual watersheds during the estimates of direct runoff volumes. In general, however, the watersheds should be used as units.

(c) *Soil Arrays.*—The relative hydrologic response of different soils or soil groups is an essential item in many engineering determinations. The following list of major soils of the United States (i.e., either of major importance locally or of major extent) is a guide to the relative base rating of any soil. A local soil not included in the list can be compared with those that are included and its relative position thus determined.

[1] "Hydrology Guide for Use in Watershed Planning," National Engineering Handbook, Section 4, Hydrology, Supplement A, U. S. Department of Agriculture, Soil Conservation Service.

[2] The text of section A-2 was largely prepared by G. W. Musgrave, Staff Specialist (Infiltration), Soil Conservation Service.

Table A-1.—*Hydrologic soil groups* [1]

GROUP A

(Includes deep sands with very little silt and clay; also deep, rapidly permeable loess)

Soil	Area or areas reported [2]
Adams	1
Alton	1
Americus	2
Ankeny	3
Arion loamy fine sand	3
Arkport	1
Arnold sandy loam	6
Arredondo	2
Ashley	6
Aspen	6
Balls loamy fine sand	5
Barnston gravelly sandy loam	6
Barth	2
Bartleson	6
Beebe	6
Bellevue	2
Berrien	3
Billingsley fine sand	4
Bingen	4
Blanton	2
Boone fine sandy loam	3
Braham	3
Brazita	4
Broward	2
Brownfield sand and loamy sand	4
Bruno sand and loamy sand	2, 4
Buckner loamy sands and sands	3
Carlisle (New York) [3]	1
Carver	1
Cashmere	6
Central	3
Chelsea sand and loamy sand	3
Chenango	1
Chetek	3
Chiefland	2
Chute	3
Cinibar silt loam	6
Cispus pumicy sandy loam	6
Clack	2
Coloma	3
Colonie	1
Colosse	1
Colton gravelly sandy loam	1
Crevasse	2
Croghan	1
Derby fine sand and loamy fine sand	4
Deschutes	6
Dolph sandy loam	6
Duelm	3
Dukes	1
Dune sand	1, 2, 5
Dwyer fine sand	5
Eatontown	1
Elliber	1
Elmira	6
Emerson	6
Enterprise loamy fine sand	4
Ephrata loamy sand and sandy loam	6
Estherville loamy sand	3
Eufala fine sand	4
Eustis	2, 4
Everett gravelly sandy loam	6
Fitch gravelly sandy loam	6
Flasher fine sand	5
Fordney loamy sand	6
Fort Meade	2
Garrison	6
Gaviota sandy loam	6
Gearhart sands	6
Geer	6
Giles very fine sandy loam	6
Glenbar	4
Goldridge fine sandy loam	6
Grayling	3
Greenwater sandy loam	6
Grimstad loamy fine sand	3
Gudrid loamy fine sand	3
Guin	2, 4
Hampden	1
Haskill fine sand	5
Hesseltine gravelly sandy loam	6
Hinckley	1
Hiwood loamy fine sand	3
Holden	6
Hoodsport gravelly loam	6
Hoosic	1
Hubbard fine sand and loamy fine sand	3
Huckabee	2
Immokalee	2
Independence	2
Indianola sandy loam	6
Isanti loamy fine sand	3
Ivie	6
Izard loamy sand	4
Jaffrey	1
Jonesville	2
Kalkaska	3
Kenney loamy fine sand	4
Kershaw	2
Kilbourne	3
Klaus gravelly loam	6
Klej	2
Kootenai	6
Lakehurst	1
Lakeland fine sand	2, 4
Lakeville gravelly loam and sandy loam	3
Lakewood	1
Lakin loamy sand	1
Lauren sandy loam	6
Leavenworth sandy loam	6
Lihen loamy fine sand	5
Lincoln fine sand and loamy fine sand	4
Lynden fine sandy loam	6
Lynndyl	6
Lystair loamy sand	6
Manchester	1
Marble sand	6
Marenisco	3
Marina sand	6
Marquette gravelly loamy sand	3
Medio fine sand	4
Menahga	3
Monteola clay	4
Moorefield	4
Morocco	3
Moro Cojo loamy sand	6
Nashville	4
Nekoosa	3
Nodaway sand	3
Nueces fine sand	4
Oakville	3
Ochlockonee loamy fine sand	4
Orelia clay loam	4
Orem	6
Orlando	2
Otisville	1
Ottawa	1, 3
Ottokee	3
Palm Beach	2
Payette	6
Pend Oreille sandy loam	6
Pentura	6
Perks	3
Peshastin stony fine sandy loam	6
Petosky	1
Plainfield	3
Plummer	2
Plymouth	1
Pound	6
Preston	6
Quincy sand and loamy sand	6
Quonset	1
Ragner fine sandy loam	6
Ratlum	1
Ravola	6
Riffe	6
Rodman	3
Roosville sand	5
Roscommon	3
Rubicon	3
Rudd	4
Rupert sand	6
Saffell	2, 4
St. Lucie	2
Santaquin	6
Sarpy loamy sand and sand	3
Selle	6
Sheppard	6
Sioux loamy sand	3
Skykomish gravelly sandy loam	6
Snoqualmie gravelly loam	6
Spanaway gravelly sandy loam	6
Spinks	3
Springdale	6
Springer loamy sands	4
Stillman	6
Syracuse	6
Tedrow	3
Thornwood gravelly loam	6
Tivoli sand and loamy sands	4, 5
Tombigbee	2
Touhey sandy loam	6
Toutle loamy sand	6

[1] Since this is the first time a hydrologic array of this kind has been prepared, this grouping is tentative and subject to change. Adjustments will be made as discrepancies are discovered and as more information and experience are gained through usage. The soil series name only is given where the textural range is narrow and the soil occurs in only one group.

[2] See areas shown on fig. A-2.

[3] Carlisle muck in Ohio is in group C.

TABLE A-1.—*Hydrologic soil groups*—Group A—Continued

Soil	*Area or areas reported* [1]
Tujunga gravelly sand	6
Tunkhannock	1
Ulen loamy fine sand	3
Valentine fine sand	5
Vebar fine sand	5
Victoria clay	4
Vilas	3
Vinton	6
Vona	5
Wardboro	6
Wasatch	6
Westport sands	6
Winchester sands	6
Wind River gravelly loam	6
Windsor	1
Yonkers	4
Zimmerman fine sand and loamy fine sand (U.S. Soil Conservation Service.)	3

[1] See areas shown on fig. A-2.

GROUP B

(Mostly sandy soils less deep than A, and loess less deep or less aggregated than A, but the group as a whole has above-average infiltration after thorough wetting)

Soil	*Area or areas reported*
Abajo	6
Abernathy	2
Ackmen	6
Acton	1
Acworth	1
Agawam	1
Aiken clay loam	6
Akron	2
Albemarle	2
Albia	1
Albion	5
Alcoa	2
Alderwood gravelly loam	6
Alexandria	3
Alicel silt loam	6
Allagash	1
Allen	2, 4
Allenwood	1
Allison	3
Alvin	3
Amarillo fine sandy loam	4
Amenia	1
Amite	2, 4
Ammon	6
Amsterdam silt loam	5
Annabella	6
Annandale	1
Anthony	4
Appling	2
Arapien	6
Arenzville	1
Arkansas fine sandy loam	4
Aroostook	1
Arveson	3
Asbury	1
Ashe	2
Ashton	1
Astoria loam	6
Athena silt loam	6
Atterberry	3
Attleboro	1
Atwood	2
Aura	1
Avery	2
Badrock gravelly loam	5
Balfour	2
Bancroft	6
Bangor	1
Bannock	6
Barbourville	2
Barnes	3
Barnstead	1
Barrington	1
Bates	4, 5
Battrick stony loam	5
Baxter	2, 4
Beatty	1
Beaver	3
Beaverton gravelly loam	5
Becket	1
Bedford	2
Beechy	2
Benedicta	1
Benewah	6
Benson	1
Berkshire	1
Bermudian	2
Berthoud silt loam	4
Bertie	2
Bertrand silt loam	3
Beulah	4
Beverly	6
Bewleyville	2
Bickleton	6
Bienville	2
Biggs	3
Biggsville	3
Billett	3
Bingham	6
Birchwood	1
Birkbeck	3
Biscay	3
Blackfoot	6
Blakely	2
Blandford	1
Blanding	6
Blodgett	5
Bodine	2
Bold	3
Bolton	2
Bonner	6
Bowie	2, 4
Bowman	1
Boxelder	6
Boyer	3
Boynton	1
Bradbury	1
Brady	3
Brandon	2, 4
Branford	1
Brassua	1
Bratton	3
Breece	5
Brennan loamy fine sand	4
Brenton	3
Bridgehampton	1
Bridgeville	1
Brimfield	1
Broadbrook	1
Brookfield	1
Brookston	3
Brownfield loamy fine sand	4
Brownlee	6
Brunswick	2
Buckland	1
Budger	6
Buncombe	4
Burke	6
Burnham	1
Burnsville	3
Burton	2
Butte stony loam	6
Bybee	2
Cabot	1
Cahaba	2, 4
Calais	1
Caldwell	6
Caleta	6
Calouse	6
Camas	6
Cameron	4
Canadian	4
Canyon	5
Capron	3
Capshaw	2
Carey	4
Caribou	1
Carmi	3
Carnegie	2
Carrington	3
Carstairs	6
Casa	4
Cascade	6
Casco	3
Castana	3
Castana-Napier complex	3
Catherine	6
Cattaraugus	1
Caylor	2
Cecil	2
Centerton	4
Chagrin	1, 3
Chalker	1
Chamokane	6
Charlton	1
Chaseburg	3
Chaseburg-Nodaway complex	3
Chattahoochee	2
Chehalis clay loam, loam, and silt loam	6
Chemawa	6
Cheshire	1
Chester	1, 2
Cheyenne	5
Chickasha	4
Chicopee	1
Chili	3
Chilmark	1
Chittenden	1
Choccolocco	2
Choctaw	4
Cisco	4
Claiborne	2
Clallam	6
Clarion	3
Clarion-Dickinson complex	3
Clarion-Nicollet complex	3
Clarksville	2, 4
Clary	3
Clearfield	3
Cleburne	2, 4
Cle Elum	6
Cleora	4

TABLE A-1.—*Hydrologic soil groups*—Group B—Continued

Soil	Area or areas reported
Clifton	2
Clinton	3
Cloquet	3
Clyde-Floyd complex	3
Clymer	1
Cobb	4
Cokesbury	1
Colburn	6
Colby	5
Colebrook	1
Colfax	2
Collbran	6
Collenston	6
Collins	2, 4
Colrain	1
Colton sandy loam	1
Conant	1
Condon	6
Conestoga	1
Congaree	2, 4
Cookeville	2
Cooper	3
Coosa	2
Copake	1
Cornwall	1
Cossayuna	1
Cotaco	2
Courtrock	6
Cowiche	6
Creedmoor	2
Creston	5
Crider	2
Crossville	2
Culleoka	2
Culpeper	2
Cumberland	2, 4
Cushman	5
Dade	2
Dakota	3
Dalhart fine sandy loam	4, 5
Danforth	1
Darnell stony fine sandy loam	4, 5
Davidson	2
Decatur	2, 4
Declo	6
Decorra	3
Dekalb	1
Dekoven	2
Delphi	6
Dewey	2
Dexter	2
Dickinson	3
Dickson	2
Dierks	2, 4
Dill	4
Disco	3
Dixie	6
Dixmont	1
Dodds	2
Dodgeville	3
Dolph loam	6
Dorchester	3
Doty	6
Dougherty	4
Douglas	6
Dover	1
Downey	6
Downs	3
Doyle	1
Dragston	2
Drake	4
Draper	6
Dresden	3
Drummer	3
Dubbs	4
Dubuque	3
Ducker	2
Duffield	1
Dunbar	2
Duplin	2
Durham	2
Dutchess	1
Duval fine sandy loam, deep phase	4
Easton	1
Ebbs	6
Edgemont	1, 2
Edneyville	2
Elco	3
El Dorado	4
Elioak	2
Elk	2, 4
Ellisforde	6
Ellison	3
Elmore	6
Elmwood	1
Emmet	3
Emory	1, 4
Endicott	6
Enfield	1
Ennis	2
Ensley	3
Enterprise very fine sandy loam	4
Essex	1
Estherville silt loam	3
Etowah	2
Eulonia	2
Ewingville	1
Faceville	2
Fairhaven	3
Faison	2
Falk	6
Fall	3
Fallbrook	6
Fallsington	2
Fannin	2
Farland	5
Farmington	1
Faunsdale	2
Fauquier	2
Fayette	3
Flanagan	3
Flathead	5
Flom	3
Florham	1
Floyd loam and silt loam	3
Forbes (undifferentiated)	3
Fossum	3
Fox	3
Foxon	1
Frankstown	1
Frederick	1
Fredon	1
Freeland	2, 4
Fruita	6
Fullerton	2
Gainesville	2
Gallion	4
Galva	3
Geary	5
Genesee	1, 3
Genola	6
Georgeville	2, 4
Gilbert	3
Gilcrest	5
Gilead	2
Gilford	3
Glendive	5
Glenelg	1, 2
Glenwood	3
Gloucester	1
Glover	1
Goldsboro	2
Goodloe	2
Grafton	1
Granby	3
Grant	4, 5
Grantsdale	5
Grantsville	6
Granville	2
Gravity	3
Green Bluff	6
Greenriver	6
Greensboro	1
Greenville	2, 4
Grimstad fine sandy loam	3
Grover	2
Groveton	1
Grygla	3
Gudrid fine sandy loam	3
Habersham	2, 4
Hackers	1
Hackettstown	1
Haddam	1
Hadley	1
Hagener	3
Hagerstown	1, 2
Halewood	2
Half Moon	5
Halsey	1
Hamblen	2
Hamburg	3
Hamilton	5
Hanceville	2, 4
Hannahatchee	4
Hartford	1
Hartland	1
Hartleton	1
Hartsells	2
Hayden	3
Hayesville	2
Haynie	3
Hayter	1, 2
Hazel	2
Hector	4
Heisler	6
Hembre	6
Hermitage	2
Hermon (Gloucester)	1
Herndon	2, 4
Hero	1
Hidalgo	4
Highland	6
Hiko Springs	6
Hillsboro	6
Hillsdale	3
Hiwassee	2, 4
Hixton	3
Hobbs	6
Hodgenville	2
Hoffman	2
Holdrege	5
Holland	6
Hollis	1
Holston	1, 2, 3, 4
Holyoke	1
Honeoye	1
Hopper	3
Houlton	1

TABLE A-1.—*Hydrologic soil groups*—Group B—Continued

Soil	Area or areas reported
Housatonic	1
Hovenweep	6
Howard	1
Hoye	6
Hubbard sandy loam	3
Hublersville	1
Hugo sandy loam (California)	6
Humphreys	2
Hunters	6
Huntington	1, 2, 3, 4
Huntsville	3
Hyattsville	2
Hyde	2
Hymon	2
Hyrum (Utah) [4]	6
Ida	3
Ina	2
Iona	3
Iron River	3
Irvington	2
Jackson	3
Jefferson	2, 4
Jenness	6
Johnston	2
Joy	3
Juliaetta	6
Kalamazoo	3
Kalmia	2, 4
Kanosh	6
Kars	1
Katama	1
Kato	3
Kaysville	6
Keith	5
Kempsville	2
Kennebec	3
Kennedy	6
Kenton	2
Kerby	6
Keyport	2
Kilburn	6
Kittitas	6
Knappa	6
Knutsen	6
Lackawanna	1
Ladoga group	3
Laidig	1
Lamont	3
Lancaster	5
Landes	3
Lanesville	1
Lansdale	1
Lansing	1
Lapine	6
Laredo	4
Larimer	5
Larkin	6
Lawrence	2
Leadvale	2
Lee	2
Leetonia	2
Lehew	1, 2
Lehew-Ashby complex	1
Leicester	1
Lempster	1
Lenox	1
Leon	2
LeSeur	3
Letort	1
Lewisberry	1
Lexington	2, 4
Lihen fine sandy loam	5
Limerick	1
Lincroft	1
Linker	2, 4
Linneus	1
Lintonia	4
Littleton	3
Lobelville	2
Lodi	2
Lolalita	6
Lolo	5
Lonoke	4
Loradale	2
Lorenzo	3
Louisa	2
Louisburg	2
Lyman	1
Lynchburg	2
Lyons	1
Machias	1
Madison	2
Madras	6
Madrid	1
Magnolia	2
Mahaska	3
Mallory	2
Manitou	2
Manor	1
Mansfield	1
Marlboro	2
Marlow	1
Marquette sandy loam	3
Marshall	3, 5
Masada	2
Masardis	1
Massilon	3
Matapeake	2
Mattapex	2
Maumee	3
Maury	2
May	4
Mayfield	6
Mayodan	2
McBeth	6
McBride	3
McCammon	6
McKenna	6
McPaul (Hornick)	3
Meda	6
Mellenthin	6
Melrose	1
Memphis	2, 4
Mendon	6
Merrimac	1
Mesa	6
Metea	3
Methow	6
Middlefield	1
Milaca	3
Miles	4
Millard	6
Mill Creek	3
Minco	4
Mindensilt	3
Minidoka	6
Minvale	2
Moffat	6
Monarda	1
Monona	3
Montalto	1
Monticello	6
Montville	1
Moody	3
Moody-Crofton-Arlon-Thurman complex	3
Morrison	1
Morton	5
Moscow	6
Moshannon	1
Moulton	6
Mt. Carroll	3
Mountview	2
Murrill	1
Muscatine	3
Muse	2
Musinia	6
Muskingum [5]	1, 2, 4
Musselshell	5
Myersville	2
Myton	6
Nantucket	1
Napier	3
Naples	6
Narcisse	6
Narragansett	1
Nashua	1
Nason	2
Nassau	1
Nebish	3
Negley	3
Nehalem	6
Nesika	6
Newberg	6
Newfield	1
Newmarket	1
Newport	1
Newton	3
Newtonia	4
Nicholson	2
Nicollet	3
Ninigret	1
Nixon	1
Noble	4
Nodaway silt loam	3
Nodaway-Napier-Colo	3
Nolichucky	2
Norfolk	2, 4
Norwood	4
Nova	6
Oasis	6
Oblon	2
Ochlockonee fine sandy loam	2, 4
Ockley	3
Okenee	2
Olequa	6
Olmitz	3
Olympic silty clay loam	6
Ona	2
Onamia	3
Onaway	3
Ondawa	1
O'Neill	3
Onslow	2
Ontario	1
Onyx	6
Ooltewah	2
Ora	2, 4
Orangeburg	2, 4
Orchard	6
Oshtemo	3
Otero	5
Othello	2
Otley	3
Ozark	4

[4] Group C in Idaho.

[5] Muskingum is in group C in Ohio.

TABLE A-1.—*Hydrologic soil groups*—Group B—Continued

Soil	Area or areas reported
Pace	2
Pack	6
Paden	2
Paiso	4
Palisade	6
Palmyra	1
Palouse	6
Paraloma	4
Parker	1
Parleys	6
Pasquotank	2
Patit Creek	6
Pawlet	1
Paxton	1
Peacham	1
Pegram	6
Pekay group	3
Pembroke	2
Penasco	4
Pennington	1
Penwood	1
Peone	6
Pequanoc	1
Perkinsville	2
Peterboro	1
Philby	3
Philo	2, 4
Pickwick	2
Pierce	3
Pinson	2, 4
Pisgah	2
Pittsfield (Nellis)	1
Pittsford	1
Pittstown	1
Plaisted	1
Pleasant Grove	6
Pocomoke	2
Podunk	1
Poe	6
Pomfret	1
Pontotoc	4
Pope	1, 2, 3, 4
Poquonock	1
Portales	4
Port Byron	3
Porters	2
Portneuf	6
Portsmouth	2
Potlatch	6
Poultney	1
Presque Isle	1
Primghar group	3
Proctor	3
Puget	6
Purgatory	6
Puyallup	6
Quakertown	1
Quandahl	3
Rabun	2
Racine	3
Rains	2
Ralston	6
Ramsey	2
Rapidan	2
Red Bay	2
Redfield	6
Red Hook	1
Red Rock	6
Reeves	4
Renslow	6
Reynolds	3
Riddle	4
Ridgebury	1
Rimer	3
Ringwood	3
Ritzville	6
Roane	2, 4
Robinsonville	2, 4
Rockaway	1
Rockingham	1
Rockmart	2
Rockton	3
Rockton-Dodgeville (chert phase)	3
Rockwell	3
Roseburg	6
Roseville	3
Royalton	1
Rumford	2
Rumney	1
Russell	3
Ruston	2, 4
Ryders	1
Sable	3
Sac	3
Saco	1
St. Agatha	1
St. Albans	1
St. Charles	3
St. George	6
St. Joe	6
St. Johns	2
St. Mary's	6
Salem	6
Salix	3
Sanpete	6
Sassafras	2
Saugatuck	1, 3
Sawmill (Illinois)	3
Saybrook	3
Scarboro	1
Schnorbush	6
Schumacher	6
Sciotoville	2
Scituate	1
Scobey	5
Scorup	6
Scranton	2
Seaton	3
Selma	3
Seneca	2
Sequatchie	1, 2
Sevy	6
Shannon	2, 4
Shapleigh	1
Sharon	3
Sharpsburg	3
Sheffield	1
Shelburne	1
Shelbyville	2
Shelton	6
Sheridan	6
Sidell	3
Sifton	6
Sigurd	6
Sinclair	6
Sioux sandy loam	3
Siskiyou	6
Sisson	3
Skerry	1
Skiyou	6
Snohomish	6
Snow	6
Sogn	5
Squapan	1
Squires	1
Starr	2
Staser	2
State	2, 4
Statesville	2
Steinsburg	2
Stephenville	4
Sterling	6
Stetson	1
Stevens	6
Stissing	1
Stockbridge	1
Storden loam	3
Strasburg	1
Stronghurst	3
Stupel	6
Sudbury	1
Sultan	6
Sumner	3
Sunset	6
Surprise	6
Sutton	1
Sverup	3
Swanton	1
Swauk	6
Tabiona	6
Talcott	3
Tallula	3
Tama	3
Tanberg	3
Tatum	2
Teller	4
Tellico	2
Terrill	3
Terry	5
Thorndike	1
Thurman-Clarion complex	3
Thurman-Marshall complex	3
Thurman-Tama complex	3
Thurmont	2
Tifton	2
Tilden	2, 4
Timpanogos	6
Timula	3
Tioga	1
Tirzah	2
Tisbury	1
Todd	3
Tokul	6
Tollgate	6
Townsbury	1
Tridell	6
Tusquitee	2
Umpqua	6
Unadilla	1
Uncompahgre	6
Valois	1
Van Buren	1
Vassar	6
Vaucluse	2
Vicksburg	2, 4
View	6
Vista	6
Volinia	3
Volney	3
Wadena	3
Wadesboro	2
Wahee	2
Waits	6
Walden	1
Wallagras	1

TABLE A-1.—*Hydrologic soil groups*—Group B—Continued

Soil	*Area or areas reported*	*Soil*	*Area or areas reported*	*Soil*	*Area or areas reported*
Walla Walla	6	Wenham	1	Winooski	1
Walpole	1	Westland loam	3	Witt	4
Warm Springs	6	Westminster	1	Woodbridge	1
Warsaw	3	Wethersfield	1	Woodside	5
Warwick (Stratham)	1	Whately	1	Woodstock (rocky)	1
Washburn	1	Wheeler	6	Woodstown	2
Washington	1	Wheeling	1, 2, 3	Woodward very fine sandy loam	4
Wassaic	1	Whitefish	5	Wooster	1, 3
Watauga	2	Whiteford	2	Woostern	1
Waukegan	3	Whitman	1	Worthen	3
Waukesha	3	Whitwell	2	Worthington	1
Waumbek	1	Wickham	2, 4		
Wauseon	3	Wilkinson	6	Yadkin	2
Waynesboro	1, 2, 4	Willamette	6		
Weaver	2	Williams	5	Zahl	5
Weeksville	2	Wilmington	3	Zane	6
Welby	6	Wiltshire	1	Zanels fine sandy loam	4

GROUP C

(Comprises shallow soils and soils containing considerable clay and colloid, though less than those of Group D. The group has below-average infiltration after pre-saturation)

Soil	*Area or areas reported*	*Soil*	*Area or areas reported*	*Soil*	*Area or area reported*
Abilene fine sandy loam	4	Belmont	1	Butler	5
Abington	3	Bennington	3	Buxton	1
Acme clay loam	4	Bentonville	3	Byars	2
Ada	6	Berks	1	Byington	2
Addison	1	Berlin	1	Cabinet	5, 6
Afton	3	Bernardston	1	Cacapon	1
Agency	6	Berwick	3	Calloway	2, 4
Agnos	4	Beryl	6	Calvin	1
Alamance	2, 4	Bethany	4	Canadice	1
Aldino	1	Bibb	2, 4	Canfield	1, 3
Algiers	3	Biddeford	1	Carbo	2
Almo	2, 4	Bieber	6	Cardington	3
Altamont	6	Big Timber	5	Carlisle muck (Ohio)[6]	3
Altavista	2, 4	Billings	4, 6	Carlton	6
Alvira	1	Binnsville	4	Catalpa clay loam	4
Amarillo clay loam	4	Bissell	6	Cathcart	6
Ambraw	3	Blacklock	6	Catron	6
Ames	3	Blockton	3	Cavode	1
Amity	6	Blount	3	Celina	3
Andres	3	Bluford	3	Centerfield	6
Angie	2	Bogota	3	Chance	6
Antelope Springs	6	Bolivar	4	Chandler	4
Applegate	6	Bono	3	Chariton	3
Ark	4	Boomer	6	Chehalis silty clay loam	6
Armagh	1	Bothwell	6	Chesterfield	2
Armuchee	2	Bowdre	4	Chewacla	2, 4
Asa	4	Brackett	4	Chilo	3
Ashby	1	Bradley	2	Chippewa	3
Ashkum	3	Brashear	2	Churchill	6
Ashwood	2	Brecknock	1	Clackamas	6
Atkins	1, 2, 3, 4	Bremer	3	Clareville	4
Auburn	6	Brenner	6	Clarksburg	1
Augusta	2, 4	Bridport	1	Clayton	6
Austin	4	Briggsdale	5	Clebit	4
Ava	3	Brinkerton	1	Cloquallum	6
Avalon	6	Brinsburg	4	Clyde	3
Baca	4	Brittain	4	Coaticook	1
Baker	6	Brood	6	Cocalalla	6
Baldock	6	Brookside	1	Colo	3
Bark River	3	Buchanan	1	Colville	6
Barrows	3	Buckhannon	1	Colwood	3
Bath	1	Bullion	6	Colyer	2
Bayside	6	Burchard	5	Commerce	4
Beasley	2	Burgin	2	Conda	6
Beaucoup	3	Burmester	6	Conowingo	1
Beauregard	4	Burrell	3		
Belinda	3	Buse	3		

[6] Carlisle is in group A in New York.

TABLE A-1.—*Hydrologic soil groups*—Group C—Continued

Soil	*Area or areas reported*
Conway	4
Cookport	1
Coral	3
Corley	3
Cornutt	6
Coston	6
Cottonwood	4
Cougar	6
Couse	6
Covington	1
Cresco	3
Crete	5
Crofton	3
Crosby	3
Croton	1
Culvers	1
Curran	3
Cut Bank	6
Cuthbert	2, 4
Dalbena	6
Dendridge	2
Danvers	5
Deary	6
Delfina	4
Delp	6
Dennis	4, 5
Denton	4
Deseret	6
Dewart	1
Dixon	4
Dixonville	6
Dulac	2, 4
Dundas	3
Dundee	4
Dunkirk	1
Dunmore	1, 2
Ecleto	4
Ector	4
Edalgo	5
Eddy	4
Eden	2
Edgington	3
Edina	3
Egam	2
Elliott	3
Ellsberry	3
Emmons	6
Enders	2, 4
Erie	1
Ernest	1
Escalante	6
Escondido	6
Esto	2
Fairview	4
Falaya	2, 4
Fallsburg	3
Fay	3
Ferron	6
Fiander	6
Fincastle	3
Fingal	6
Fitchville	3
Flint	2, 4
Florence	5
Flowell	6
Floyd (plastic till variant)	3
Floyd (thin surface variant)	3
Fort Pierce	6
Freesoil	3
Fremont	1
Frio	4
Fulton loam	1, 3

Soil	*Area or areas reported*
Galvin	6
Gara	3
Garwin	3
Garwin-like	3
Gem	6
Gerald	4
Gilpin	1
Glasgow	6
Glencoe	3
Glenford	3
Glenville	1, 2
Goessel	5
Goldston	4
Gooch	6
Gooding	6
Goose Creek	6
Gordon	6
Gosport	3
Grail	5
Grantsburg	3
Greendale	2, 4
Grenada	2, 4
Gresham	1
Groseclose	2
Grundy	3, 5
Guelph	3
Guernsey	1
Hack	6
Haig	3
Hansel	6
Harley	4
Harpster	3
Harwood	6
Hastings	5
Hatchie	2
Heath	5
Heflin	4
Helena	2
Helmer	6
Herrick	3
Hesson	6
Hockley	4
Hollister silt loam and clay loam	4
Hollywood	2
Hosmer	3
Houlka	2
Huckleberry	6
Hudson	1
Hugo silt loam (Oregon)	6
Humeston	3
Hyrum (Idaho) [7]	6
Idana	5
Inkom	6
Inman	2
Isabella	3
Ivins	6
Izagora	2, 4
Johnsburg	2, 4
Josephine	6
Jura	6
Kasson, thick A_2 variant	3
Kasson-like, nearly level variant	3
Kasson-like silt loam	3
Katy	4
Keene (Coshocton, Ohio, Station)	3
Kelso	6
Kendaia	1
Kettleman	6
Kilchus	6
Kings	3

[7] Group B in Utah.

Soil	*Area or area reported*
Kirkham	6
Kirvin	2, 4
Kistler	1
Kitsap	6
Klamath	6
Koster	6
Krum	4
Labette	5
La Luz	4
Lamonta	6
Lanark	6
Langford	1
Lassen	6
Latah	6
Laughlin	6
Leeper	2
Lehigh	1
Leland	6
Leshe	4
Lewisville	4
Lickdale (West Virginia and Southeast) [8]	1, 2
Lima	1
Lindley	3
Lindside	2, 3, 4
Lisbon	3
Little Timber	5
Litz	1, 2
Livingston	1
Lobdell	3
Lockerby	6
Logan	6
Lorain	3
Lordstown	1
Loring	2, 4
Los Osos	6
Loudonville	3
Lucas	1
Lumberton	3
Luray	3
Luverne	2, 4
Macon	2
Madalin	1
Manassa	6
Manastash	6
Mansker	4
Mantachie	2, 4
Marcus	3
Mardin	1
Marengo	3
Marias	5
Marion	3
Marshan	3
Massie	3
Mayhew	4
Maytown	6
Mazeppa	1
McCornick	6
McKey	6
Medio loamy fine sand	4
Meigs	1, 3
Melbourne	6
Melvin	1, 2, 4
Mench	1
Mercer	2
Meriba	4
Metuchen	1
Mhoon	2, 4
Midnay	5
Millbrook	3
Miller clay loam and silt loam	4

[8] Group D in Arkansas.

TABLE A-1.—*Hydrologic soil groups*—Group C—Continued

Soil	*Area or areas reported*
Mimbres	4
Mission	6
Modale	3
Modoc	6
Monongahela	1, 2, 3, 4
Monroeville	3
Montevallo	2
Montgomery	3
Montwel	6
Morley	3
Moro Bay	2, 4
Morris	1
Morrow	6
Munising	3
Muskingum [9]	3
Muskogee	4
Nacimiento	6
Natchitoches	2
Negrett	4
Newark	4
Newell	6
Nez Perce	6
Nimrod	4
Nixa	4
Nolo	1
Norma	6
Norwich	1
Nunn	5
O'Brien	3
Oconee	3
Odessa	1
O'Fallon	3
Okoboji	3
Olivier	4
Olmstead	3
Olympic clay loam	6
Omboy	1
Oquaga	1
Orange	2
Orrville	3
Orwell	1
Panton	1
Papakating	3
Parsippany	1
Passaic	1
Patrick	4
Paul	6
Pearman	2
Pearson	4
Pedernales	4
Penn	1, 2
Pershing	3
Petrolia	3
Pewamo	3
Pharo	6
Pheba	2, 4
Pilot Rock	6
Pinoyer	6
Poganeab	6
Pond Creek	4
Potter	4
Powell	6
Power	6
Prentiss	2, 4
Prior	5
Providence	2, 4
Pullman	4
Rahway	1
Ramona	6
Rankin	3
Ravenna	3
Rayne	1
Readington	1
Reagan	4
Reaville	1
Red Bayou	4
Redmond	6
Regent	5
Renfro	4
Rhinebeck	1
Richfield	5
Richland	4
Richview	3
Ridgecrest	6
Roellen	2
Rolfe	3
Rossmoyne	3
Ruckles	6
Sagemore	6
St. Paul	4
Salkum	6
Sango	2
Santa	6
Santa Lucia	6
Sauvie	6
Savage	5
Savannah	2, 4
Sawyer	4
Saxon	6
Scantic	1
Schapville (Iowa) [10]	3
Schoharie	1
Selah	6
Sequoia	2
Seymour	3
Shawnee	4
Shay	6
Shelby	3, 5
Shelmadine	1
Shelocta	1
Shiloh	3
Shirley	1
Shoals	3
Shooks	3
Shoreham	1
Shubute	2, 4
Shumaker	6
Sierra	6
Simas	6
Sims	3
Sites	6
Skanee	3
Skumpah	6
Skyberg	3
Sleath	3
Snake	6
Sodus	1
Southwick	6
Sperry	3
Spur	4
Staley	6
Standfield	6
Starks	3
Steinauer	5
Stidham	4
Storden clay	3
Stough	2, 4
Stowbridge	4
Stoy	3
Suffield	1
Summit	4, 5
Sumter	2, 4
Sutherlin	6
Sweeney clay loam (California) [11]	6
Sweet	6
Tabor	4
Taft	2, 4
Taintor	3
Talladega	2, 4
Tarrant	4
Tate	2, 4
Taylorsflat	6
Taylorsville	6
Teas	1, 2
Thackery	3
Thatcher	6
Thatuna	6
Thomasville	4
Tilsit	1, 2, 3, 4
Tippah	2
Tisch	6
Tishomingo	4
Toledo	1, 3
Tonawanda	1
Trappist	2
Trask	6
Trexler	1
Troy	1
Trumbull	1
Tupelo	2
Tuscumbia	2
Tyler	1, 2, 3, 4
Ucolo	6
Upshur	1, 2
Uvada	6
Uvalde	4
Valier	5
Vallecitos	6
Vance	2
Vandalia	1
Varna	3
Vega	6
Veneta	6
Vergennes	1
Vian	4
Virden	3
Virgil	3
Volusia	1
Waha	6
Waldock	6
Wardwell	6
Warne	2
Watchung	1
Waterville	6
Watogua	4
Webb	4
Webster	3
Webster-Nicollet complex	3
Weikert	1
Weinbach	3
Weld	5
Weller	3
Westland (heavier than loam—Ohio)	3
Westmoreland	1, 2, 3
Weston	2
Westville	3
Westwater	6
Weymouth	4
Wharton	1

[9] Group B in areas 1, 2, 4.

[10] Group D in Illinois.

[11] Group D in Oregon.

TABLE A-1.—*Hydrologic soil groups*—Group C—Continued

Soil	Area or areas reported	Soil	Area or areas reported	Soil	Area or areas reported
Wheelon	6	Wolftever	2	Wyatt	1
Whippany	1	Woodrow	6	Xenia	3
Wilkes	2	Woodscross	6	Zaneis silt loam	4
Windthorst fine sandy loam	4	Woodward clay loam	4	Zapata	4
Wingville	6	Woolper	2	Zoar	1
Winterset	3	Worsham	2		

GROUP D

(Includes mostly clays of high swelling percent; but the group also includes some shallow soils with nearly impermeable subhorizons near the surface)

Soil	Area or areas reported
Abbott	6
Abilene silty clay loam	4
Acme silty clay loam	6
Airport	6
Albaton	3
Alligator	4
Almont	4
Arno	4
Arvada	5
Avonburg	3
Badlands	5
Battlecreek	6
Bear Lake	6
Beaumont	4
Bell	4
Bellingham	6
Benjaman	6
Bergland	3
Bernard	4
Bethel	3
Black Canyon	6
Blago	4
Blencoe	3
Bow	6
Bramwell	6
Brennan fine sandy loam	4
Brewer	4
Brooklyn	3
Bryce	3
Bude	4
Calumet	4
Carroll	4
Catalpa clay	4
Cayucos	6
Chamber	6
Chastain	2, 4
Cherokee	4, 5
Cleveland	4
Chilcott	6
Chipeta	6
Christianburg	6
Churchill	6
Cisne	3
Clarence	3
Clarinda	3
Clatsop	6
Clermont	3
Climax	6
Colbert	2, 4
Coldwater	3
Colp	3
Colt	4
Concord	6
Condit	3
Conover	3
Corydon	1, 4
Courtney	6
Cove	6
Crawford	4
Crockett	4
Crowley	4
Cusick	6
Darwin	3
Day	6
Dayton	6
Defiance	3
Delmita	4
Denmark	6
Denny	3
Denrock	3
DeSoto	3
Diablo	6
Dowellton	2
Drain	6
Drummond	4
Duggins	6
Dunning	2, 4
Duval fine sandy loam [12]	4
Earl	4
Edge	4
Edna	4
Ellsworth	3
Erath	4
Eutaw	4
Fargo	3
Flora	3
Foard	4
Foley (Wynne)	4
Forestdale	4
Fulton silty clay	3
Garfield	6
Garrett	4
Gasconade	4
Gay	3
Gee	6
Geiger	4
Ginat	3
Gore	4
Grande Ronde	6
Gunnison	6
Guthrie	4
Harding	6
Hardy	6
Harrisburg	6
Henneke	6
Henry	2, 3, 4
Holcomb	6
Hollister silty clay loam	4
Hortman	4
Houston	2, 4
Huey	3
Hunt	2, 4
Iredell	2
Irving	4
Jacob	3
Kaufman	4
Kent	3
Kipling	4
Kirkland	5
Kopiah	6
Kosmos	6
Lacamas	6
Ladysmith	5
Lafe	4
Lagonda	3
Lahonton	6
Lake Charles	4
Leaf	2, 4
Lebanon	4
Le Flore	4
Lela	4
Letha	6
Lickdale (Arkansas) [13]	4
Lightning	4
Lismas	5
Lucien	4
Lufkin	4
Lukin	3
Luton	3
Mahoning	3
Manila	6
McKamie	4
Mellor	6
Menefee	6
Meskell	6
Middle	6
Miguel	4
Miller clay and silty clay loam	4
Milroy	3
Modena	6
Moenkopie	6
Monee	3
Montara	6
Montoya	4
Morse	4
Mullins	4
Myatt	2, 4
Napa (Luton) clay (black alkali phase)	3
Napanee	3
Naturita	6
Navajo	6
Navasota	4

[12] Duval fine sandy loam (deep phase) is in group B.

[13] Group C in Southeast.

TABLE A-1.—*Hydrologic soil groups*—Group D—Continued

Soil	Area or areas reported
Neola	6
Neosha	5
Nester	3
Nevada	4
Niota	3
Northdale	6
Odin	6
Okaw	3
Oktibbeha	2, 4
Onawa	3
Ontonagon	3
Orson	3
Osage	5
Osceola	3
Page	4
Parlette	6
Parsons	4, 5
Pauchuta	4
Paulding	3
Pavant	6
Pawnee	5
Payne	4
Payson	6
Pecos	4
Penjur	4
Perry	4
Persayo	4, 6
Phillips	5
Pickford	3
Pierre	5
Placentia	6
Pledger	4
Portland	4
Post	5
Poteau	4
Pottsville	2
Prescott	4
Racoon	3
Rafael	6
Randall	4
Rantoul	3
Ravalli	5
Reardan	6
Reed	6
Rhoades	5
Richmond	6
Rinard	3
Ritteman	3
Roanoke	2, 4
Robertsville	2, 4
Roselms	3
St. Clair	3
Saltair	6
San Antonio	4
San Joaquin	6
San Saba	4
Sawmill silty clay loam (Iowa)	3
Schapville (Illinois)[14]	3
Sebring	3
Selkirk	3
Sharkey clay	2, 4
Shavano	6
Slights	6
Snowville	6
Souva	4
Spur	4
Susquehanna	2, 4
Sweeney (Oregon)[15]	6
Tabler	4
Talbott	2, 4
Taloka	4
Taylor	3
Telehina	4
Terminal	6
Tillman	4
Timpahute	6
Tortugas	6
Tower	6
Trenton	6
Trinity	4
Trumbull	3
Tunica	4
Una	2, 4
Valden	2, 4
Valara	4
Vanderdasson	6
Verhalen	4
Vernon	4
Veyo	6
Virgin River	6
Wabash	3
Wade	5
Wadsworth	3
Wapato	6
Watsonville	6
Waverly	2, 4
Weehadkee	4
Weir	3
West Point	4
Whatcom	6
Whiteson	6
White Store	2
Whitson	3
Wilcox	4
Wilson	4
Windthorst clay loam	4
Wing (valley fill)	4
Winslow	6
Wyemouth	4
Wrightsville	4
Wynoose	3
Yoncalla	6
Zaca	6
Zook	3

[14] Group C in Iowa.
[15] Group C in California.

The soil names will be found on agricultural soil maps which are available for a large portion of the United States. Section 82 discusses the availability of agricultural soil maps and surveys, and the type of information that is shown in them. Figure 28 shows the extent of published agricultural soil mapping in the United States.

The array is based on the premise that soils of similar profile characteristics (particularly depth, texture, organic matter content, structure, and degree of swelling when saturated) will respond in essentially similar manner under a long storm of appreciable intensity. In making comparisons it is assumed that the soils have minimum cover (bare); maximum swelling has taken place; and the applied rainfall exceeds potential infiltration. Since different types of infiltrometers are known to give different magnitudes of f_c[3] for the same soils, it is necessary to avoid using data from diverse techniques of measurement in developing the array. One soil cannot be placed on the basis of one technique of measurement and another soil placed according to a different method. All types of information are justifiably used in placing a soil in its proper relative position among other soils.

The measurement of infiltration rates through the artificial application of water, as by "rainfall simulators" or by one of several procedures for flooding the soil surface without rain impact, has been a common procedure for comparing soils or the effects of vegetation. These procedures are valuable for the *relative* comparisons they provide, but they are not satisfactory as a means of expressing infiltration rates quantitatively.

Extensive comparisons under well replicated and controlled conditions show:

(1) Rainfall simulators providing rainfall impact and turbidity of surface water give lower rates for the same soils and vegetation

[3] Minimum infiltration rate.

than do flooding types. (Thus types F and FA infiltrometers give lower rates than do tubes or rings.)

(2) Infiltrometers of larger ground area give lower rates than do those covering a smaller area where the proportion of border effects is greater.

(3) Generally, infiltrometers also tend to give larger rates than those derived from watersheds.

Conversion factors for adjusting the results from different techniques to an equivalent base have been found to be impractical.

To overcome these difficulties, use is made of the extensive knowledge of soil profile characteristics possessed by the soil scientists. On this basis the major soils of the country are placed in an array, certain members of which already have been rated by watershed studies.

When the major soils of the United States are arranged in proper relative order, the range of f_c will begin with tight clays—essentially zero rates under these conditions—and extend to the maximum rates of deep, well-aggregated silts or to those of the deep sands such as are found in sandhill areas.

The curve of the array is essentially that of figure A–1. A possible normal range of variation due to variation within a given soil by depth, structure, or texture is expressed by the dash line—25 percent above and 25 percent below the mean. The mean is indicated by the heavy line. On this curve a number of points have already been established and additional ones will be forthcoming from evaluation of watersheds, analyses of existing research data, and new experimental work. Thus the magnitude of the curve can be fixed on the basis of minimum watershed performance. The problem of converting different kinds of infiltrometer data to a watershed basis is thus solved.

It is clear that the curve presenting this array of United States soils can be regarded as a base or "floor" upon which may be superimposed the effects, for example, of lesser soil moisture, varying kinds of vegetation, and the accumulation with time of organic matter derived from better vegetation. Supplementary information on the divergent effects of soil moisture on sands, clays, or laterites, of the aggregating effects of organic matter on these different kinds of soil, and the resultant effect on infiltration, will aid in the practical application of the basic soils rating.

The major soils in each of four hydrologic groups are listed in table A–1. The area or areas in which each has been reported are indicated by the numeral or numerals following each soil name and which are keyed to the map, figure A–2.

Acknowledgments are due the numerous soil scientists and soil correlators who have so freely provided basic information on the physical properties of these soil profiles. It is their detailed knowledge that has made possible this array of about 2,000 major soils of the continental United States.

A–3. *Land Use and Treatment Classes.*—(a) *Purpose.*—These classes are used in the preparation of hydrologic soil-cover complexes (sec. A–4), which in turn are used in estimating direct runoff. Types of land use and treatment are classified on a flood runoff-producing basis. The greater the ability of a given land use or treatment to increase total retention, the lower it is on a flood runoff-production scale. Land use or treatment types not described here may be classified by interpolation, as discussed in section A–4.

(b) *Crop Rotations.*—The sequence of crops on a watershed must be evaluated on the basis of its hydrologic effects. Rotations range from *poor* (or weak) to *good* (or strong) largely in proportion to the amount of dense vegetation in the rotation. *Poor rotations* are those in which a row crop or small grain is planted in the same field year after year. A poor rotation may combine row crops, small grains, or fallow, in various ways. *Good rotations* will contain alfalfa or other close-seeded legumes or grasses, to improve tilth and increase infiltration. For example, a 2-year rotation of wheat and fallow may be a good rotation for crop production where low annual rainfall is a limiting factor, but hydrologically it is a poor rotation.

(c) *Native Pasture and Range.*—Three conditions are used, based on hydrologic considerations, not on forage production. *Poor pasture or range* is heavily grazed, has no mulch, or has plant cover on less than about 50 percent of the area. *Fair pasture or range* has between about 50 and 75 percent of the area with plant cover and is not heavily grazed. *Good pasture or range* has more than about 75 percent of the area with plant cover, and is lightly grazed.

(d) *Farm Woodlots.*—The classes are based on

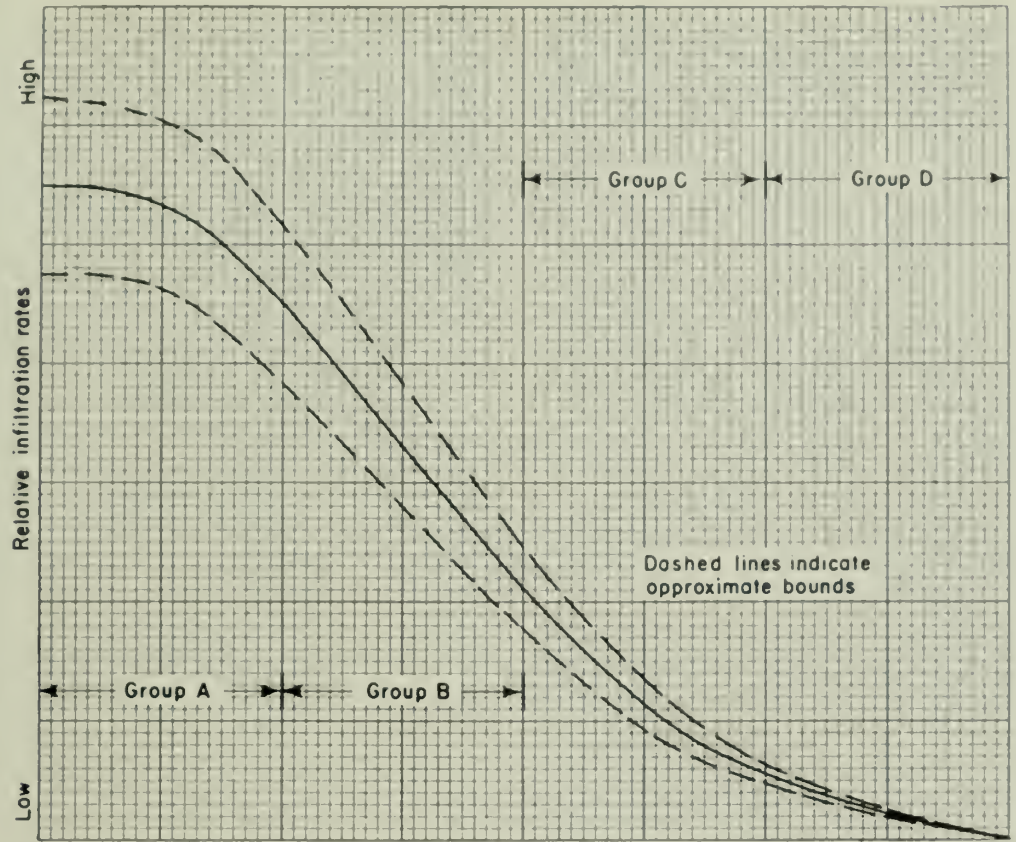


Figure A-1. **Relative infiltration rates of hydrologic soil groups. (U.S. Soil Conservation Service.)**

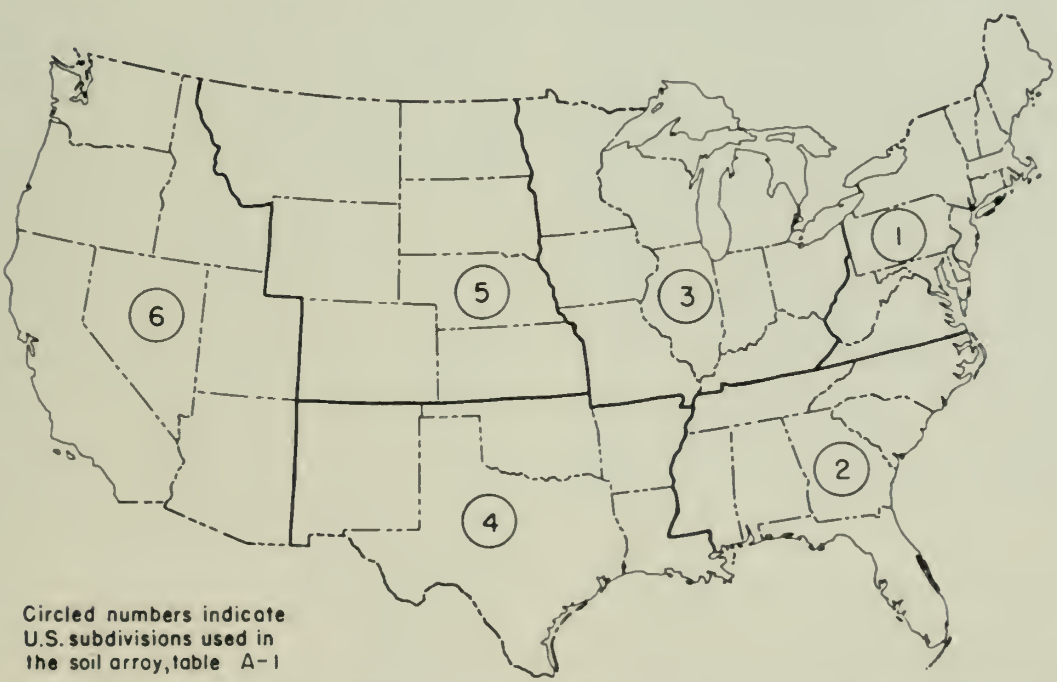


Figure A-2. **Index map to location of major soil groups listed in table A-1. (U.S. Soil Conservation Service.)**

hydrologic factors, not on timber production. *Poor woodlots* are heavily grazed and regularly burned in a manner that destroys litter, small trees, and brush. *Fair woodlots* are grazed but not burned. These woodlots may have some litter, but usually these woods are not protected. *Good woodlots* are protected from grazing so that litter and shrubs cover the soil.

(e) *Commercial Forest.*—The hydrologic condition classes are determined on the basis of depth and quality of litter, humus, and compactness of humus. The U.S. Forest Service procedure for determining the classes is given in section A–4.

(f) *Miscellaneous.*—Usually only very small parts of a watershed are in farmsteads, roads, and urban areas. When this is so, the areas may be included with one of the other land use cover types (such as fallow or small grain) in the computation of runoff (see sec. A–4).

Provision is made in table A–2 (sec. A–4) for farmsteads and roads. These land uses are generalized, since they vary so much. Where it is necessary to work with more detail (as sometimes in a very small watershed, or with superhighway, airport, or urban areas) the impervious areas are considered an individual class with 100 percent runoff, and the remaining land uses are handled as usual (see sec. A–4).

(g) *Straight-Row Farming.*—This class includes up-and-down and cross-slope farming in straight rows. In areas of 1 or 2 percent slope, cross-slope farming in straight rows is almost the same as contour farming. Where the proportion of cross-slope farming is believed to be significant, it may be classed halfway between straight-row and contour farming in the table A–2.

(h) *Contouring.*—Contour furrows used with small grains and legumes are made while planting, are generally small, and tend to disappear due to climatic action. Contour furrows, and beds on the contour, as used with row crops are generally large. They may be made in planting and later reduced in size by cultivation, or they may be insignificant after planting and become large from cultivation. Average conditions are used in table A–2.

Surface runoff reductions due to contour farming are greater as land slopes decrease. The curve numbers for contouring shown in table A–2 were obtained using data from experimental watersheds having slopes of 3 to 8 percent.

Contour furrows in pasture or range land are usually of the permanent type. Their dimensions and spacing generally vary with climate and topography. Table A–2 considers average conditions in the Great Plains.

(i) *Terracing.*—Terraces may be graded, open-end level, or closed-end level. The effects of graded and open-end level terraces are considered in table A–2, and the effects of both contouring and the grass waterway outlets are included.

TABLE A–2.—*Runoff curve numbers for hydrologic soil-cover complexes*

[FOR WATERSHED CONDITION II, AND $I_a=0.2\ S$][3]

Land use or cover	Treatment or practice	Hydrologic condition for infiltrating	Hydrologic soil group			
			A	B	C	D
Fallow	SR		77	86	91	94
Row crops	SR	Poor	72	81	88	91
	SR	Good	67	78	85	89
	C	Poor	70	79	84	88
	C	Good	65	75	82	86
	C&T	Poor	66	74	80	82
	C&T	Good	62	71	78	81
Small grain	SR	Poor	65	76	84	88
	SR	Good	63	75	83	87
	C	Poor	63	74	82	85
	C	Good	61	73	81	84
	C&T	Poor	61	72	79	82
	C&T	Good	59	70	78	81
Close-seeded legumes [1] or rotation meadow.	SR	Poor	66	77	85	89
	SR	Good	58	72	81	85
	C	Poor	64	75	83	85
	C	Good	55	69	78	83
	C&T	Poor	63	73	80	83
	C&T	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	C	Poor	47	67	81	88
	C	Fair	25	59	75	83
	C	Good	6	35	70	79
Meadow (permanent).		do	30	58	71	78
Woods (farm woodlots).		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads			59	74	82	86
Roads (dirt)[2]			72	82	87	89
(hard surface).[2]			74	84	90	92

[1] Close-drilled or broadcast.
[2] Including right-of-way.
[3] See sec. A–5.
SR = Straight row.
C = Contoured.
T = Terraced.
C&T = Contoured and terraced.

(U.S. Soil Conservation Service.)

Closed-end level terraces should be handled like contour furrows.

A–4. Hydrologic Soil-Cover Complexes. (a) *Purpose.* Table A–2 combines soil groups and land use and treatment classes into *hydrologic soil-cover complexes.* The numbers show the relative value of the complexes as direct runoff-producers (see sec. A–5). The higher the number, the greater the amount of direct runoff to be expected from a storm.

(b) *Table A–2.* The table was prepared using data from gaged watersheds with known soils and cover. Storm rainfall was plotted versus direct runoff for annual floods and other significant floods. The curve of figure A–4 best fitting the plotted points was determined, and its number was used to obtain an average curve number (II-curve) for table A–2. Related numbers for above-average (III-curve) and below-average (I-curve) points were similarly developed.

Curve numbers for several soil-cover complexes were estimated or computed from relations developed in the work, since actual hydrologic data were not available for all given complexes.

(c) *Forest Service Procedure.* Table A–3 shows curve numbers developed by the U.S. Forest Service. These numbers are used in hydrologic evaluations of commercial or national forest. Figure A–3 gives a nomograph by which the hydrologic condition class of forest is estimated. The following definitions are used with figure A–3.

(1) *Litter.*—This includes the fermentation layer. It consists of undecomposed dead vegetal material including grasses, forbs, leaves, needles, twigs, bark, etc. It varies in depth with season, being thinnest in the late winter. The fermentation or F layer consists of partly decomposed litter, with the origin still recognizable.

(2) *Humus.*—This includes either the H layer of *mor* (also known as *duff* or raw humus), or the A layer (otherwise called *mull*), in which the organic matter is incorporated in the mineral soil.

When the condition class is obtained on figure A–3, it is used only with part I of table A–3. Part II of that table gives tentative special values prepared by the Forest Service for certain forest-range areas in the western United States.

(d) *Determination of Curve Numbers for Mixed Areas.*—Table A–4 shows the process by which a weighted soil-cover complex number is obtained for areas having several soil-cover complexes. The example area is in a B soils group. The weighted number can be more easily obtained by accumulative multiplication on a calculator.

TABLE A–3.—*Runoff curve numbers for hydrologic soil-cover complexes*

I. COMMERCIAL OR NATIONAL FOREST, FOR WATERSHED CONDITION II, AND $I_a = 0.2S$

Hydrologic condition class	Hydrologic soil group			
	A	B	C	D
I Poorest	56	75	86	91
II Poor	46	68	78	84
III Medium	36	60	70	76
IV Good	26	52	62	69
V Best	15	44	54	61

II. FOREST-RANGE AREAS IN WESTERN UNITED STATES, FOR WATERSHED CONDITION III, AND $I_a = 0.2S$

Cover	Condition	Soil groups			
		A	B	C	D
Herbaceous	Poor		90	94	97
	Fair		84	92	95
	Good		77	86	93
Sagebrush	Poor		81	90	
	Fair		66	83	
	Good		55	66	
Oak-Aspen	Poor		80	86	
	Fair		60	73	
	Good		50	60	
Juniper	Poor		87	93	
	Fair		73	85	
	Good		60	77	

(Note that this table is for condition III.)

(Data supplied by U.S. Forest Service, June 1956, to U.S. Soil Conservation Service.)

TABLE A–4.—*Sample computation, weighting of hydrologic soil-cover complex numbers*

Complex	Curve number	Percent of area	Number times percent
Row crop, straight row, good rotation	78	56.2	4,384
Legumes, contoured, good rotation	69	37.5	2,588
Meadow, permanent	58	6.3	365
Total		100	7,337

Weighted number $= \frac{7,337}{100} = 73.37$ (U.S. Soil Conservation Service.)

Round-off to 73.

A–5. Estimation of Direct Runoff From Rainfall.—(a) *General.*—The method uses three variables in

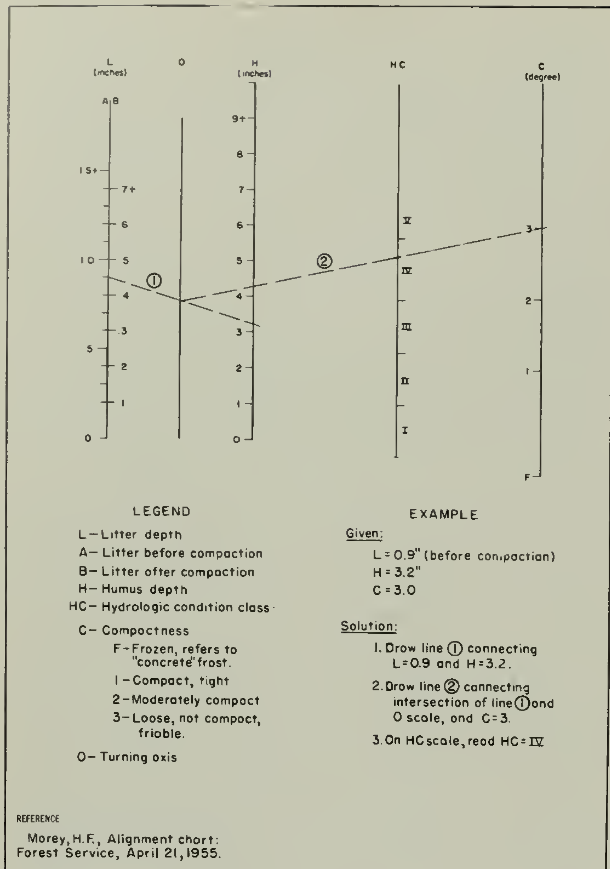


Figure A–3. **Chart for determining hydraulic condition of forest and woodland. (U.S. Soil Conservation Service.)**

estimating runoff: rainfall, antecedent moisture condition, and the hydrologic soil-cover complex.

(b) *Runoff Equation.*—The curves of figure A–4 are obtained using the equation:

$$Q=\frac{(P-0.2S)^2}{P+0.8S} \tag{1}$$

where:

Q=direct runoff, in inches

P=storm rainfall, in inches, and

S=maximum potential difference between P and Q, in inches, at time of storm's beginning.

Equation (1) is derived by starting with the proportion:

$$\frac{P-Q}{S}=\frac{Q}{P} \tag{2}$$

where $\frac{P-Q}{S}$ is visualized as the ratio of actual to potential difference between P and Q, and $\frac{Q}{P}$ is visualized as the ratio of actual to potential runoff.

Solving for Q gives:

$$Q=\frac{P^2}{P+S} \tag{3}$$

Equation (3) is useful under conditions where there is a possibility of runoff whenever there is rainfall. For the condition that $Q=0$ at a value of P greater than zero, use of an initial abstraction, I_a, is required (see the diagram on fig. A–4). With the condition that I_a cannot be greater than P, equation (2) then becomes:

$$\frac{(P-I_a)-Q}{S}=\frac{Q}{(P-I_a)} \tag{4}$$

And solving for Q gives:

$$Q=\frac{(P-I_a)^2}{(P-I_a+S)} \tag{5}$$

Since S includes I_a, an empirical relation between the variables can be developed to simplify equation (5). Data from watersheds in various parts of the country give:

$$I_a=0.2\ S \tag{6}$$

Substituting ($0.2S$) for I_a in equation (5) gives equation (1).

Equation (5) can be further expanded to recognize other given conditions or factors. However, such expansions in themselves do not mean greater accuracy in runoff estimating, since this would require that the terms expressing additional factors be of a high order of accuracy.

(c) *Significance of S.*—Plottings of direct runoff, Q, versus storm rainfall, P, on natural watersheds show that Q approaches P as P continues to increase in the storm. The same data show that $(P-Q)$ approaches a constant as P continues to increase. The quantities can be shown together as in equation (2), above, and it is apparent that the constant S is the maximum difference $(P-Q)$ that could occur for the given storm and watershed conditions. The proportion can be made more complex (as discussed above), but not all the additional factors can be related to S.

The variable S is therefore a maximum potential $(P-Q)$. During a storm, the actual $(P-Q)$ that

occurs is limited by either soil-water storage or an infiltration rate as P increases. The maximum potential $(P-Q)$ or S, therefore, is dependent on soil-water storage and the infiltration rates of a watershed.

(d) *Significance of I_a.*—The insert on figure A–4 shows that I_a is equal to the rainfall that occurs before runoff starts. Physically, I_a consists principally of interception, infiltration, and surface storage. Equation (6), which relates I_a to S, is based on data from large and small watersheds in various parts of the country. Further refinement of equation (6) is not recommended, since the data needed to break I_a into components of interception, infiltration, and surface storage are seldom available on a watershed basis. For the same reason, adjustment of the coefficient 0.2 in equation (6) is not recommended.

(e) *System of Curve Numbering.*—For convenience in interpolation, the curves of figure A–4 are numbered from 100 to zero. The numbers are related to S as follows:

$$\text{Curve number}=\frac{1{,}000}{10+S} \qquad (7)$$

A curve for the case $I_a=0$, equation (3), is displaced to the right for the case $I_a=0.2S$, equation (1), by the amount of $0.2S$. Therefore, the curve numbers given in table A–2 should be used only with figure A–4 or with equation (1).

(f) *Antecedent Conditions.*—The amount of rainfall in a period of 5 to 30 days preceding a particular storm is referred to as antecedent rainfall, and the resulting condition of the watershed in regard to potential runoff is referred to as an antecedent condition. In general, the heavier the antecedent rainfall, the greater the direct runoff that occurs from a given storm. The effects of infiltration and evapo-transpiration during the

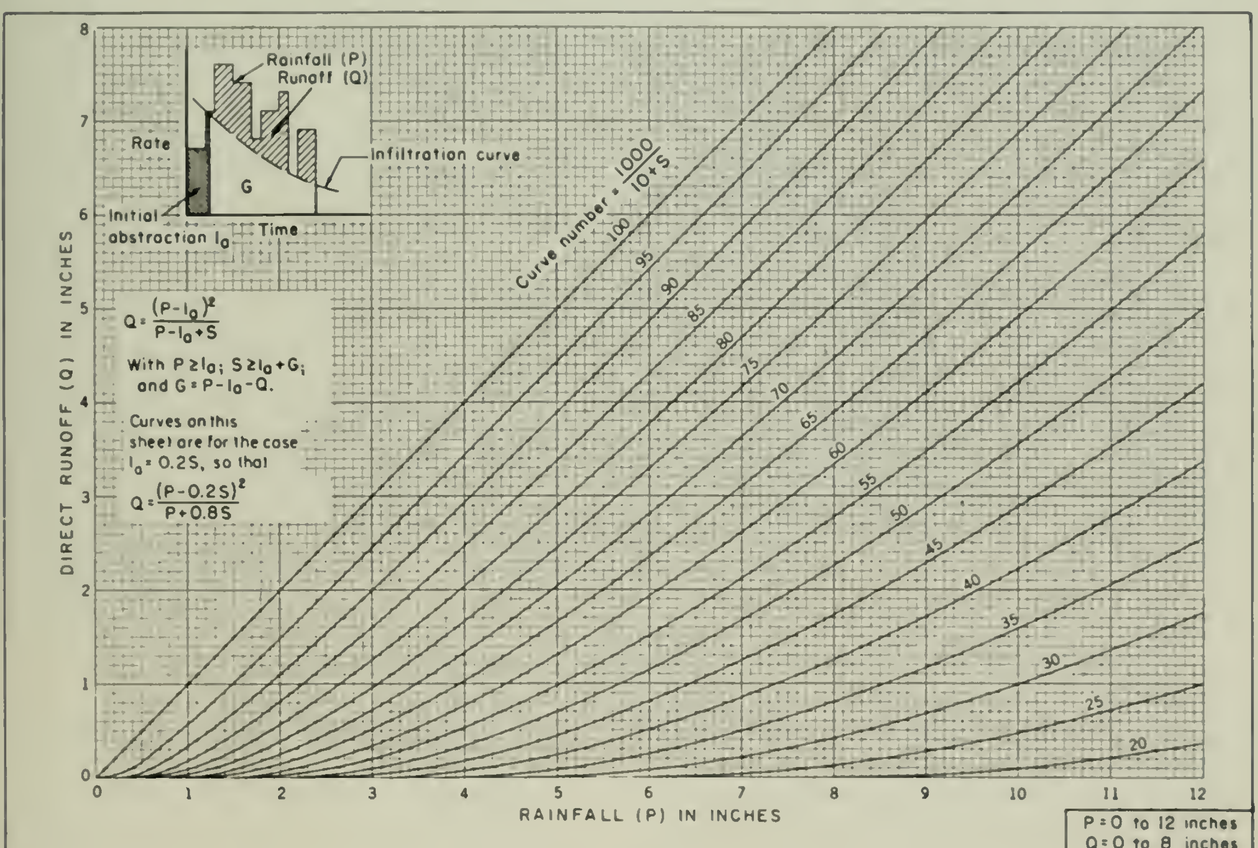


Figure A–4. Solution of runoff equation, $Q=\frac{(P-0.2S)^2}{P+0.8S}$. (Sheet 1 of 2.) (U.S. Soil Conservation Service.)

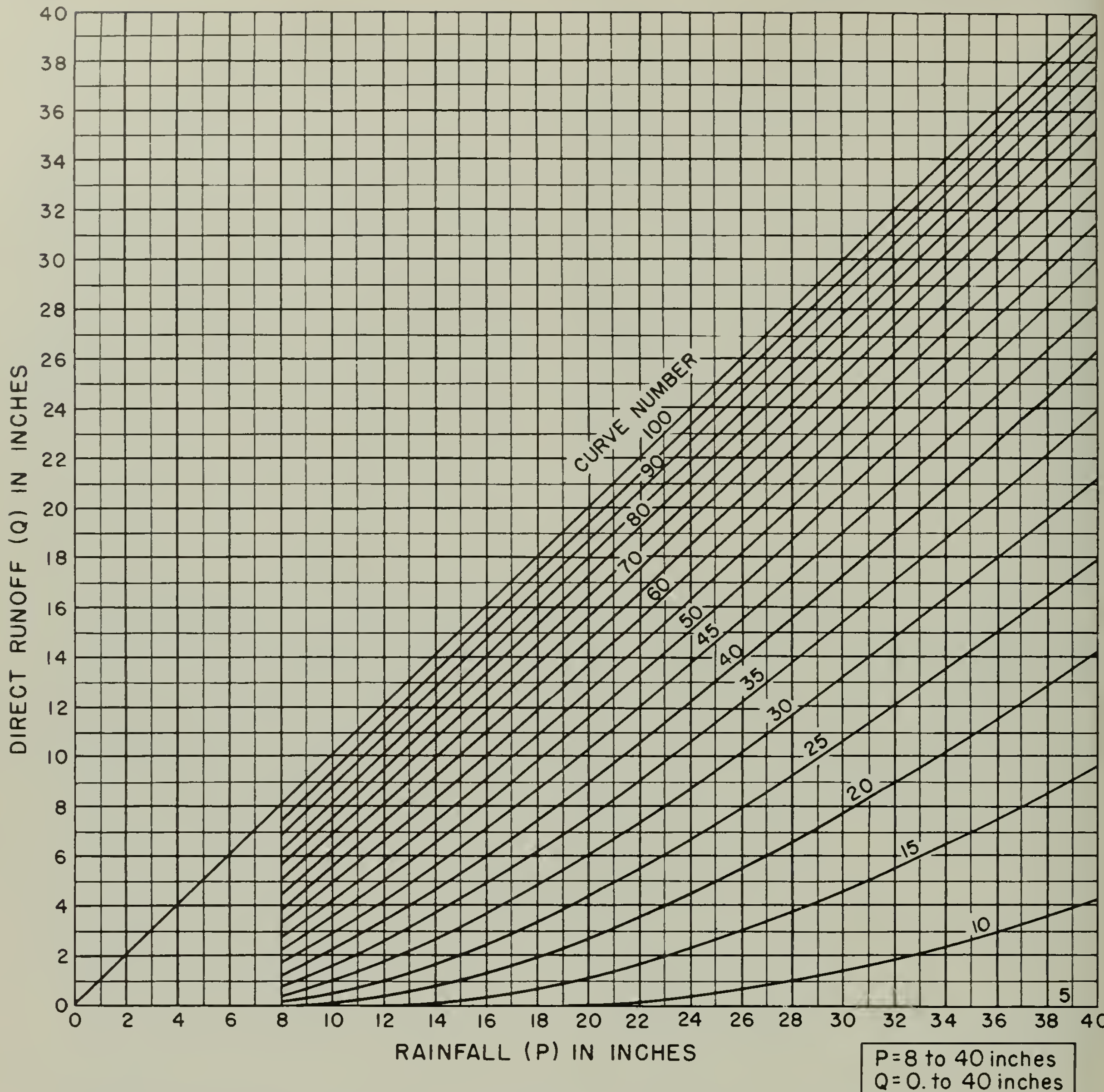


Figure A-4. **Solution of runoff equation,** $Q=\frac{(P-0.2S)^2}{P+0.8S}$. **(Sheet 2 of 2.) (U.S. Soil Conservation Service.)**

antecedent period are also important, as they may increase or lessen the effect of antecedent rainfall.

Because of the difficulties of determining antecedent storm conditions from data normally available, the conditions are reduced to the following three cases:

Condition I.—A condition of watershed soils where the soils are dry but not to the wilting point, and when satisfactory plowing or cultivation takes place. (This condition is *not* considered applicable to the design flood computation methods presented in this text.)

TABLE A-5. — *Conversions and constants*

FOR THE CASE $I_a = 0.2\ S$

1	2	3	4	5
Curve number for condition II	Corresponding curve numbers for		*S* values [1]	Curve [1] originates where *P* =
	Condition I	Condition III		
100	100	100	0	0
95	87	99	.526	.10
90	78	98	1.11	.22
85	70	97	1.76	.35
80	63	94	2.50	.50
75	57	91	3.33	.67
70	51	87	4.29	.86
65	45	83	5.38	1.08
60	40	79	6.67	1.33
55	35	75	8.18	1.64
50	31	70	10.00	2.00
45	27	65	12.2	2.44
40	23	60	15.0	3.00
35	19	55	18.6	3.72
30	15	50	23.3	4.66
25	12	45	30.0	6.00
20	9	39	40.0	8.00
15	7	33	56.7	11.34
10	4	26	90.0	18.00
5	2	17	190.0	38.00
0	0	0	Infinity	Infinity

[1] For curve number in col. 1. (U. S. Soil Conservation Service.)

Condition II.—The average case for *annual floods*, that is, an average of the conditions which have preceded the occurrence of the maximum annual flood on numerous watersheds.

Condition III.—When heavy rainfall or light rainfall and low temperatures have occurred during the 5 days previous to the given storm, and the soil is nearly saturated.

The numbers on table A-2 and table A-3A are for the average watershed condition, condition II. The numbers on table A-3B are for the nearly saturated condition, condition III. Curve numbers for one antecedent condition may be converted to a different antecedent condition by the use of table A-5. For example, the computation given in table A-4 results in a condition II curve number of 73. The corresponding curve numbers for condition I and condition III can be obtained from columns 2 and 3 of table A-5, by interpolation. The curve numbers for condition I and condition III are 55 and 89, respectively.

Hydraulic Computations

C. J. HOFFMAN AND J. M. LARA [1]

A. HYDRAULIC FORMULAS

B-1. ***Lists of Symbols and Conversion Factors.***—The following list includes symbols used in hydraulic formulas given in chapters VIII and IX and in this appendix. Standard mathematical notations and symbols having only very limited applications have been omitted.

Symbol	*Description*
A,a	An area; area of a surface; cross-sectional area of flow in an open channel; cross-sectional area of a closed conduit
a_g	Gross area of a trashrack
a_n	Net area of a trashrack
B	Width of a siphon throat
b	Bottom width of a channel
C	A coefficient; coefficient of discharge
C_d	Coefficient of discharge through an orifice
C_i	Coefficient of discharge for an ogee crest with inclined upstream face
C_o	Coefficient of discharge for a nappe-shaped ogee crest designed for an H_o head
C_s	Coefficient of discharge for a partly submerged crest
D	Diameter; conduit diameter; height of a rectangular conduit or passageway; height of a square orifice
$\overline{D}$	"Drop number" parameter for defining the dimensions of a straight drop spillway, $\overline{D}=\frac{q^2}{gY^3}$
d	Depth of flow in an open channel; height of an orifice or gate opening
d_c	Critical depth
d_f	Depth of the pool under a free overfall nappe
d_H	Depth for high (subcritical) flow stage (alternate to d_L)
d_j	Height of a hydraulic jump (difference in the conjugate depths)
d_L	Depth for low (supercritical) flow stage (alternate to d_H)
d_m	Mean depth of flow
d_{m_c}	Critical mean depth
d_n	Depth of flow measured normal to channel bottom
d_s	Depth of scour below tailwater in a plunge pool
d_t	Depth of flow in a chute at tailwater level
E	Energy
E_m	Energy of a particle of mass
F	Froude number parameter for defining flow conditions in a channel, $F=\frac{v}{\sqrt{gd}}$
F_t	Froude number parameter for flow in a chute at the tailwater level
f	Friction loss coefficient in the Darcy-Weisbach formula, $h_f=\frac{fL}{D}\frac{v^2}{2g}$
G	Drop in water surface level in a reach of a natural channel
g	Acceleration due to the force of gravity
H	Head over a crest; head on center of an orifice opening; head on the bottom of a culvert entrance; head difference at a gate (between the upstream and downstream water surface levels)
H_A	Absolute head above a datum plane, in channel flow
H_{AT}	Probable minimum atmospheric pressure at the site under consideration
H_a	Head above a section in the transition of a drop inlet spillway
H_1	Head measured to bottom of an orifice opening
H_2	Head measured to top of an orifice opening
h	Head; height of baffle block; height of end sill
h_a	Approach velocity head
h_b	Head loss due to bend
h_c	Head loss due to contraction
H_D	Head from reservoir water surface to water surface at a given point in the downstream channel

[1] Engineer, Spillway and Outlet Works Section, and Engineer, Sedimentation Section, Bureau of Reclamation.

Symbol	Description
h_d	Difference in water surface level, measured from reservoir water surface to the downstream channel water surface
H_E	Specific energy head
H_{E_c}	Specific energy head at critical flow
H_e	Total head on a crest, including velocity of approach
h_e	Head loss due to entrance
h_{ex}	Head loss due to expansion
h_f	Head loss due to friction
Δh_f	Incremental head loss due to friction
h_g	Head loss due to gates or valves
h_L	Head losses from all causes
Σh_{L_u}	Sum of head losses upstream from a section
Δh_L	Incremental head loss from all causes
$\Sigma(\Delta h_L)$	Sum of incremental head losses from all causes
H_o	Design head over ogee crest
h_o	Head measured from the crest of an ogee to the reservoir surface immediately upstream, not including the velocity of approach (crest shaped for design head H_o)
h_r	Reduction of pressure head due to inlet contraction
H_s	Total head over a sharp-crested weir
h_s	Head over a sharp-crested weir, not including velocity of approach
h_S	Priming head on a siphon spillway
h_{SA}	Subatmospheric pressure head
H_T	Total head from reservoir water surface to tailwater, or to center of outlet of a free-discharging pipe
h_t	Head loss due to trashrack
h_v	Velocity head; head loss due to exit
h_{v_c}	Critical velocity head
h_{v_S}	Velocity head at throat of siphon spillway
h_{v_t}	Velocity head at tailwater level
K	A constant factor for various equations; a coefficient
k	A constant
K_a	Abutment contraction coefficient
K_b	Bend loss coefficient
K_c	Contraction loss coefficient
K_d	Conveyance capacity factor in the Manning formula, $K_d=\frac{1.486}{n}ar^{2/3}$
K_e	Entrance loss coefficient
K_{ex}	Expansion loss coefficient
K_g	Gate or valve loss coefficient
K_L	A summary loss coefficient for losses due to all causes
K_p	Pier contraction coefficient
K_r	Coefficient of pressure reduction due to inlet contraction
K_t	Trashrack loss coefficient
K_v	Velocity head loss coefficient
L	Length; length of a channel or a pipe; effective length of a crest; length of a hydraulic jump; length of a stilling basin; length of a transition
ΔL	Incremental length; incremental channel length
L_I, L_{II}, L_{III}	Stilling basin lengths for different hydraulic jump stilling basins
L'	Net length of a crest
L_B	Length of a basin for a straight drop spillway; length of a slotted grating dissipator basin
L_d	Distance from the upstream face of an overflow weir to the start of a hydraulic jump in a straight drop spillway stilling basin
L_G	Length of a slotted grating dissipator
L_m	Length of a meandering reach in a natural channel
L_p	Distance from the upstream face of an overflow weir to the point of impingement on the basin floor of a straight drop spillway
L_s	Length of a straight reach in a natural channel
M	Momentum
M_d	Momentum in a downstream section
M_u	Momentum in an upstream section
ΔM	Difference in momentum between successive sections
m	Mass
N	Number of piers on an overflow crest; number of slots in a slotted grating dissipator
n	Exponential constant used in equation for defining crest shapes; coefficient of roughness in the Manning equation
P	Approach height of an ogee weir; hydrostatic pressure of a water prism cross section
p	Unit pressure intensity; unit dynamic pressure on a spillway floor; wetted perimeter of a channel or conduit cross section
Q	Discharge; volume rate of flow
ΔQ	Incremental change in rate of discharge
q	Unit discharge
Q_c	Critical discharge
q_c	Critical discharge per unit of width
Q_i	Average rate of inflow
Q_o	Average rate of outflow
R	Radius; radius of a cross section; crest profile radius; vertical radius of curvature of the channel floor profile; radius of a terminal bucket profile
r	Hydraulic radius; radius of abutment rounding; radius of rounding of a culvert inlet opening
R_b	Radius of a bend in a channel or pipe
R_c	Radius of curvature at the crest of a siphon throat

Symbol	Description
R_s	Radius of curvature at the summit of a siphon throat; radius of a circular sharp-crested weir
S	Storage
ΔS	Increment of storage
s	Friction slope in the Manning equation; spacing
s_b	Slope of the channel floor, in profile
s_{ws}	Slope of the water surface
T	Tailwater depth; width at the water surface in a cross section of an open channel
T_{max}	Limiting maximum tailwater depth
T_{min}	Limiting minimum tailwater depth
t	Time
Δt	Increment of time
T_s	Tailwater sweep-out depth
$T.W.$	Tailwater, tailwater depth
U	A parameter for defining flow conditions in a closed waterway, $U=\frac{v}{\sqrt{gD}}$
v	Velocity
Δv	Incremental change in velocity
v_a	Velocity of approach
v_c	Critical velocity
v_s	Velocity at the crest of a siphon throat
v_t	Velocity of flow in a channel or chute, at tailwater depth
W	Weight of a mass; width of a stilling basin
w	Unit weight of water; width of a culvert entrance; width of a slot for a slotted grating dissipator; width of chute and baffle blocks in a stilling basin
x	A coordinate for defining a crest profile; a coordinate for defining a channel profile; a coordinate for defining a conduit entrance
Δx	Increment of length
x_c	Horizontal distance from the break point, on the upstream face of an ogee crest, to the apex of the crest
x_s	Horizontal distance from the vertical upstream face of a circular sharp-crested weir to the apex of the undernappe of the overflow sheet
Y	Drop distance measured from the crest of the overflow to the basin floor, for a free overfall spillway
y	A coordinate for defining a crest profile; a coordinate for defining a channel profile; a coordinate for defining a conduit entrance
$\bar{y}$	Depth from water surface to the center of gravity of a water prism cross section
Δy	Difference in elevation of the water surface profile between successive sections in a side channel trough
y_c	Vertical distance from the break point, on the upstream face of an ogee crest, to the apex of the crest
y_s	Vertical distance from the crest of a circular sharp-crested weir to the apex of the undernappe of the overflow sheet
Z	Elevation above a datum plane
ΔZ	Elevation difference of the bottom profile between successive sections in an open channel
z	Ratio, horizontal to vertical, of the slope of the sides of a channel cross section
α	A coefficient; angular variation of the side wall with respect to the structure centerline
θ	Angle from the horizontal; angle from vertical of the position of an orifice; angle from the horizontal of the edge of the lip of a deflector bucket

Table B-1 presents conversion factors most frequently used by the designer of small dams to convert from one set of units to another; for example, to convert from cubic feet per second to acre-feet. Also included are some basic conversion formulas such as the ones for converting flow for a given time to volume.

B-2. *Flow in Open Channels.* (a) *Energy and Head.*—If it is assumed that streamlines of flow in an open channel are parallel and that velocities at all points in a cross section are equal to the mean velocity v, the energy possessed by the water is made up of two parts: kinetic (or motive) energy and potential (or latent) energy. Referring to figure B-1, if W is the weight of a mass m, the mass possesses Wh_2 foot-pounds of energy with reference to the datum. Also, it possesses Wh_1 foot-pounds of energy because of the pressure exerted by the water above it. Thus, the potential energy of the mass m is $W(h_1+h_2)$. This value is the same for each particle of mass in the cross section. Assuming uniform velocity, the kinetic energy of m is $W\left(\frac{v^2}{2g}\right)$.

Thus, the total energy of each mass particle is:

$$E_m=W\left(h_1+h_2+\frac{v^2}{2g}\right) \tag{1}$$

Applying the above relationship to the whole discharge Q of the cross section in terms of the unit weight of water w,

$$E=Qw\left(d+Z+\frac{v^2}{2g}\right) \tag{2}$$

where E is total energy per second at the cross section.

TABLE B–1.—*Conversion factors and formulas*

[To reduce units in column 1 to units in column 4, multiply column 1 by column 2
To reduce units in column 4 to units in column 1, multiply column 4 by column 3]

CONVERSION FACTORS

Column 1	Column 2	Column 3	Column 4
LENGTH			
In	2.54 0.0254	0.3937 39.37	Cm. M.
Ft	0.3046	3.2808	M.
Miles	1.609	0.621	Km.
AREA			
Sq. in	6.4516	0.1550	Sq. cm.
Sq. m	10.764	.0929	Sq. ft.
Sq. miles	27.8784×10^{6} 640.0 30.976×10^{5} 2.59	0.3587×10^{-7} $.15625\times10^{-2}$ $.3228\times10^{-6}$ .386	Sq. ft. Acres (1 section). Sq yd. Sq. km.
Acre	43,560.0 4,046.9 4,840.0	0.22957×10^{-4} $.2471\times10^{-3}$ $.2066\times10^{-3}$	Sq. ft. Sq. m. Sq. yd.
VOLUME			
Cu. ft	1,728.0 7.4805 6.2321	0.5787×10^{-3} .13368 .16046	Cu. in. Gal. Imperial gal.
Cu. m	35.3145 1.3079	0.028317 .76456	Cu. ft. Cu. yd.
Gal	231.0 3.7854	0.4329×10^{-2} .26417	Cu. in. Liters.
Million gal	133,681.0 3.0689	0.74805×10^{-5} .32585	Cu. ft. Acre-ft.
Imperial gal	1.2003	0.83311	Gal.
Acre-in	3,630.0	$.27548\times10^{-3}$	Cu. ft.
Acre-ft	1,233.5 43,560.0	0.81071×10^{-3} $.22957\times10^{-4}$	Cu. m. Cu. ft.
In. on 1 sq. mile	232.32×10^{4} 53.33	0.43044×10^{-6} .01875	Cu. ft. Acre-ft.
Ft. on 1 sq. mile	278.784×10^{5} 640.0	0.3587×10^{-7} $.15625\times10^{-2}$	Cu. ft. Acre-ft.
VELOCITY AND GRADE			
Miles/hr	1.4667	0.68182	Ft./sec.
M./sec	3.2808 2.2369	.3048 .44704	Ft./sec. Miles/hr.
Fall in ft./mile	189.39×10^{-6}	5.28×10^{3}	Fall/ft.

CONVERSION FACTORS

Column 1	Column 2	Column 3	Column 4
FLOW			
Cu. ft./sec. (second-feet) (sec.-ft.).	60.0	0.016667	Cu. ft./min.
	86,400.0	$.11574\times10^{-4}$	Cu. ft./day.
	31.536×10^{6}	$.31709\times10^{-7}$	Cu. ft./yr.
	448.83	$.2228\times10^{-2}$	Gal./min.
	646,317.0	$.15472\times10^{-5}$	Gal./day.
	1.98347	.50417	Acre-ft./day.
	723.98	$.13813\times10^{-2}$	Acre-ft./365 days.
	725.78	$.13778\times10^{-2}$	Acre-ft./366 days.
	55.54	.018005	Acre-ft./28 days.
	57.52	.017385	Acre-ft./29 days.
	59.50	.016806	Acre-ft./30 days.
	61.49	.016262	Acre-ft./31 days.
	50.0	.020	Miner's inch in Idaho, Kans., Nebr., N. Mex., N. Dak., S. Dak., and Utah.
	40.0	.025	Miner's inch in Ariz., Calif., Mont., Nev., and Oreg.
	38.4	.026042	Miner's inch in Colo.
	35.7	.028011	Miner's inch in British Columbia.
	0.028317	35.31	Cu. m./sec.
	1.699	.5886	Cu. m./min.
	0.99173	1.0083	Acre-in./hr.
Cu. ft./min	7.4805 10,772.0	0.13368 $.92834\times10^{-4}$	Gal./min. Gal./day.
10^{6} gal./day	1.5472 694.44 3.0689	0.64632 $.1440\times10^{-2}$ .32585	Cu. ft./sec. Gal./min. Acre-ft./day.
In. depth/hr	645.33	0.15496×10^{-2}	Sec.-ft./sq. mile.
In. depth/day	26.889 53.33	0.03719 .01878	Sec.-ft./sq. mile. Acre-ft./sq. mile.
Sec.-ft./sq. mile	1.0413 1.0785 1.1157 1.1529 13.574 13.612	0.96032 .92720 .89630 .86738 .073668 .073467	In. depth/28 days. In. depth/29 days. In. depth/30 days. In. depth/31 days. In. depth/365 days. In. depth/366 days.
Acre-ft./day	226.24 20.17 19.36	0.442×10^{-2} .0496 .0517	Gal./min. Miner's inch in Calif. Miner's inch in Colo.
Gal./sec	5.347 5.128	0.187 .195	Miner's inch in Calif. Miner's inch in Colo.
PERMEABILITY			
Meinzer (gal./day through 1 sq. ft. under unit gradient).	48.8	0.02049	Bureau of Reclamation (cu. ft./yr. through 1 sq. ft. under unit gradient).

TABLE B-1.—*Conversion factors and formulas*—Continued

CONVERSION FACTORS

Column 1	Column 2	Column 3	Column 4
POWER AND ENERGY			
Hp.	555.0 0.746 6,535. 42.4 1.0	0.18182×10^{-2} 1.3405 0.15303×10^{-3} .0236 1.0	Ft.-lb./sec. Kw. Kw.-hr./yr. B.t.u./min. Sec.-ft. falling 8.8 ft.
Hp.-hr.	0.746 198.0×10^{4} 2,545.0	1.3405 0.505×10^{-6} $.393\times10^{-3}$	Kw.-hr. Ft.-lb. B.t.u.
Kw.	8,760.0 737.56 11.8 3,412.0	0.11416×10^{-3} $.1354\times10^{-2}$ .0846 $.29308\times10^{-3}$	Kw.-hr./yr. Ft.-lb./sec. Sec.-ft. falling 1 ft. B.t.u./hr.
Kw.-hr.	0.975	1.025	Acre-ft. falling 1 ft.
B.t.u.	778.0 0.1×10^{-3} to $.834\times10^{-4}$	0.1285×10^{-2} 10,000 to 12,000	Ft.-lb. Lb. of coal
PRESSURE			
Ft. water at max. density	62.425 0.4335 .0295 .8826 773.3	0.01602 2.3087 33.93 1.133 0.1293×10^{-2}	Lb./sq. ft. Lb./sq. in. Atm. In. Hg at 30° F. Ft. air at 32° F. and atm. pressure.
Ft. avg. sea water	1.026	0.9746	Ft. pure water.
Atm., sea level, 32° F.	14.697	.06804	Lb./sq. in.
Millibars	295.299×10^{-4} 75.008×10^{-3}	33.863 1.3331	In. Hg. Mm. Hg.
Atm.	29.92	33.48×10^{-3}	In. Hg.
WEIGHT			
P.p.m.	0.00136 .0584 8.345	735.29 17.123 0.1198	Tons/acre-ft. Gr./gal. Lb./10^{6} gal.
Lb.	7.0×10^{3}	0.14286×10^{-3}	Gr.
Gm.	15.432	.064799	Gr.
Kg.	2.2046	.45359	Lb.
Lb. water at 39.1° F.	27.6812 0.11983 .09983 .453617 .01602 .01560	0.03612 8.345 10.016 2.204 62.425 64.048	Cu. in. Gal. Imperial gal. Liters. Cu. ft. pure water. Cu. ft. sea water.
Lb. water at 62° F.	0.01604 .01563	62.355 63.976	Cu. ft. pure water. Cu. ft. sea water.

FORMULAS

VOLUME

Average depth in inches, or acre-inch per acre

$$=\frac{(\text{cu. ft./sec.})\ (\text{hr.})}{\text{acres}}$$

$$=\frac{(\text{gal./min.})\ (\text{hr.})}{(450)\ (\text{acres})}$$

$$=\frac{(\text{miner's in.})\ (\text{hr.})}{(40^{*})\ (\text{acres})}$$

*Where 1 miner's in. = 1/40 sec.-ft.

Use 50 where 1 miner's in. = 1/50 sec.-ft.

Conversion of inches depth on area to sec.-ft.

$$\text{sec.-ft.}=\frac{(645)\ (\text{sq. miles})\ (\text{in. on area})}{(\text{time in hr.})}$$

POWER AND ENERGY

$$\text{hp.}=\frac{(\text{sec.-ft.})\ (\text{head in ft.})}{8.8}$$

$$=\frac{(\text{sec.-ft.})\ (\text{pressure in lb./sq. in.})}{3.8}$$

$$=\frac{(\text{gal./min.})\ (\text{head in ft.})}{3{,}960}$$

$$=\frac{(\text{gal./min.})\ (\text{pressure in lb./sq. in.})}{1{,}714}$$

$$\text{b. hp.}=\frac{\text{water hp.}}{\text{pump efficiency}}$$

kw.-hr./1,000 gal. pumped/hr.

$$=\frac{(\text{head in ft.})\ (0.00315)}{(\text{pump efficiency})\ (\text{motor efficiency})}$$

Kw.-hr. = (plant efficiency) (1.025) (head in ft.) (water in acre-ft.)

$$\text{Load factor}=\frac{(\text{kw.-hr. in time } t)}{(\text{kw. peak load})\ (\text{time } t \text{ in hr.})}$$

SEDIMENTATION

Tons/acre-ft. = (unit weight cu. ft.) (21.78)

Tons/day = (sec.-ft.) (p.p.m.) (0.0027)

TEMPERATURE

$$°\text{C}=\frac{5}{9}(°\text{F}-32°)\qquad °\text{F}=\frac{9}{5}°\text{C}+32°$$

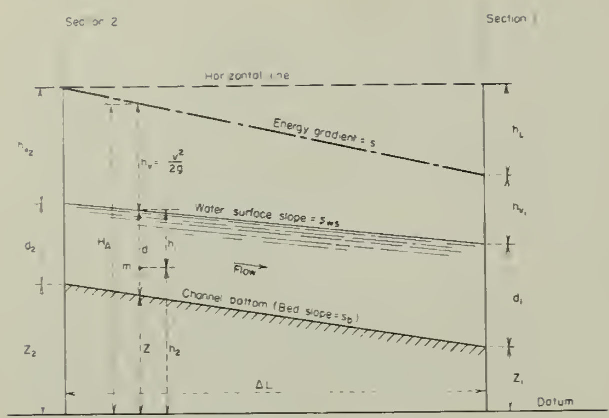


Figure B–1. **Characteristics of open-channel flow.**

The portion of equation (2) in the parentheses is termed the absolute head, and is written:

$$H_A=d+Z+\frac{v^2}{2g} \tag{3}$$

Equation (3) is called the Bernoulli equation.

The energy in the cross section referred to the bottom of the channel is termed the specific energy. The corresponding head is referred to as the specific energy head and is expressed as:

$$H_E=d+\frac{v^2}{2g} \tag{4}$$

Where $Q=av$, equation (4) can be stated:

$$H_E=d+\frac{Q^2}{2ga^2} \tag{5}$$

For a trapezoidal channel where b is the bottom width and z defines the side slope, if q is expressed as $\frac{Q}{b}$ and a is expressed as $d(b+zd)$, equation (5) becomes:

$$H_E=d+\frac{q^2}{2gd^2\left(1+\frac{zd}{b}\right)^2} \tag{6}$$

Equation (5) is represented in diagrammatic form on figure B–2 to show the relationships between discharge, energy, and depth of flow in an open channel. The diagram is drawn for several values of unit discharge in a rectangular channel.

It can be seen that there are two values of d, d_H and d_L, for each value of H_E, except at the point where H_E is minimum, where only a single value exists. The depth at energy $H_{E_{min}}$ is called the critical depth, and the depths for other values of H_E are called alternate depths. Those depths lying above the trace through the locus of minimum depths are in the subcritical flow range and are termed subcritical depths, while those lying below the trace are in the supercritical flow range and are termed supercritical depths.

Figure B–3 plots the relationships of d to H_E as stated in equation (6), for various values of unit discharge q and side slope z. The curves can be used to quickly determine alternate depths of flow in open channel spillways.

(b) *Critical Flow.*—Critical flow is the term used to describe open channel flow when certain relationships exist between specific energy and discharge and between specific energy and depth. As indicated in section B–2(a) and as demonstrated on figure B–2, critical flow terms can be defined as follows:

(1) *Critical discharge.*—The maximum discharge for a given specific energy, or the discharge which will occur with minimum specific energy.

(2) *Critical depth.*—The depth of flow at which the discharge is maximum for a given specific energy, or the depth at which a given discharge occurs with minimum specific energy.

(3) *Critical velocity.*—The mean velocity when the discharge is critical.

(4) *Critical slope.*—That slope which will sustain a given discharge at uniform critical depth in a given channel.

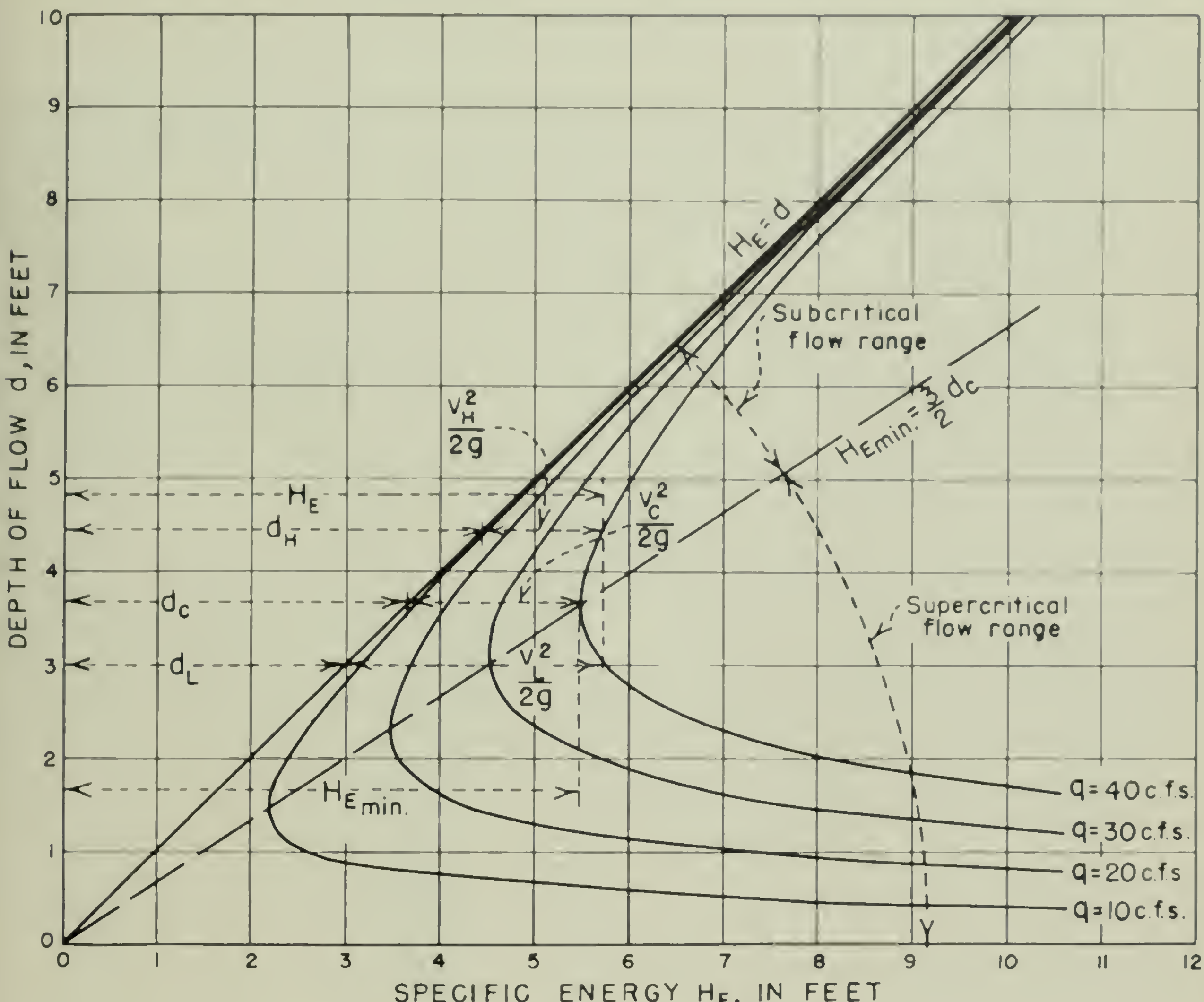
(5) *Subcritical flow.*—Those conditions of flow for which the depths are greater than critical and the velocities are less than critical.

(6) *Supercritical flow.*—Those conditions of flow for which the depths are less than critical and the velocities are greater than critical.

More complete discussions of the critical flow theory in relationship to specific energy are given in most hydraulic textbooks [1, 2, 3, 4, 5].[2] The relationship between cross section and discharge which must exist in order that flow may occur at the critical stage is:

$$\frac{Q^2}{g}=\frac{a^3}{T} \tag{7}$$

[2] Numbers in brackets refer to items in the bibliography, sec. B–10.



$$H_E = d + \frac{V^2}{2g} = d + \frac{q^2}{2gd^2}$$ where q = discharge per unit width.

$$d_c = \left(\frac{q_c}{\sqrt{g}}\right)^{\frac{2}{3}} = \frac{2}{3} H_{E_{min.}}$$ where d_c = critical depth

q_c = critical discharge per unit width

$H_{E_{min.}}$ = minimum energy content

Figure B–2. **Depth of flow and specific energy for rectangular section in open channel.**

where:

a = cross sectional area in square feet, and

T = water surface width in feet.

Since $Q^2 = a^2 v^2$, equation (7) can be written:

$$\frac{v_c^2}{2g} = \frac{a}{2T} \qquad (8)$$

Also, since $a = d_m T$, where d_m is the mean depth of flow at the section, and $\frac{v_c^2}{2g} = h_{v_c}$, equation (8) can be rewritten:

$$h_{v_c} = \frac{d_{m_c}}{2} \qquad (9)$$

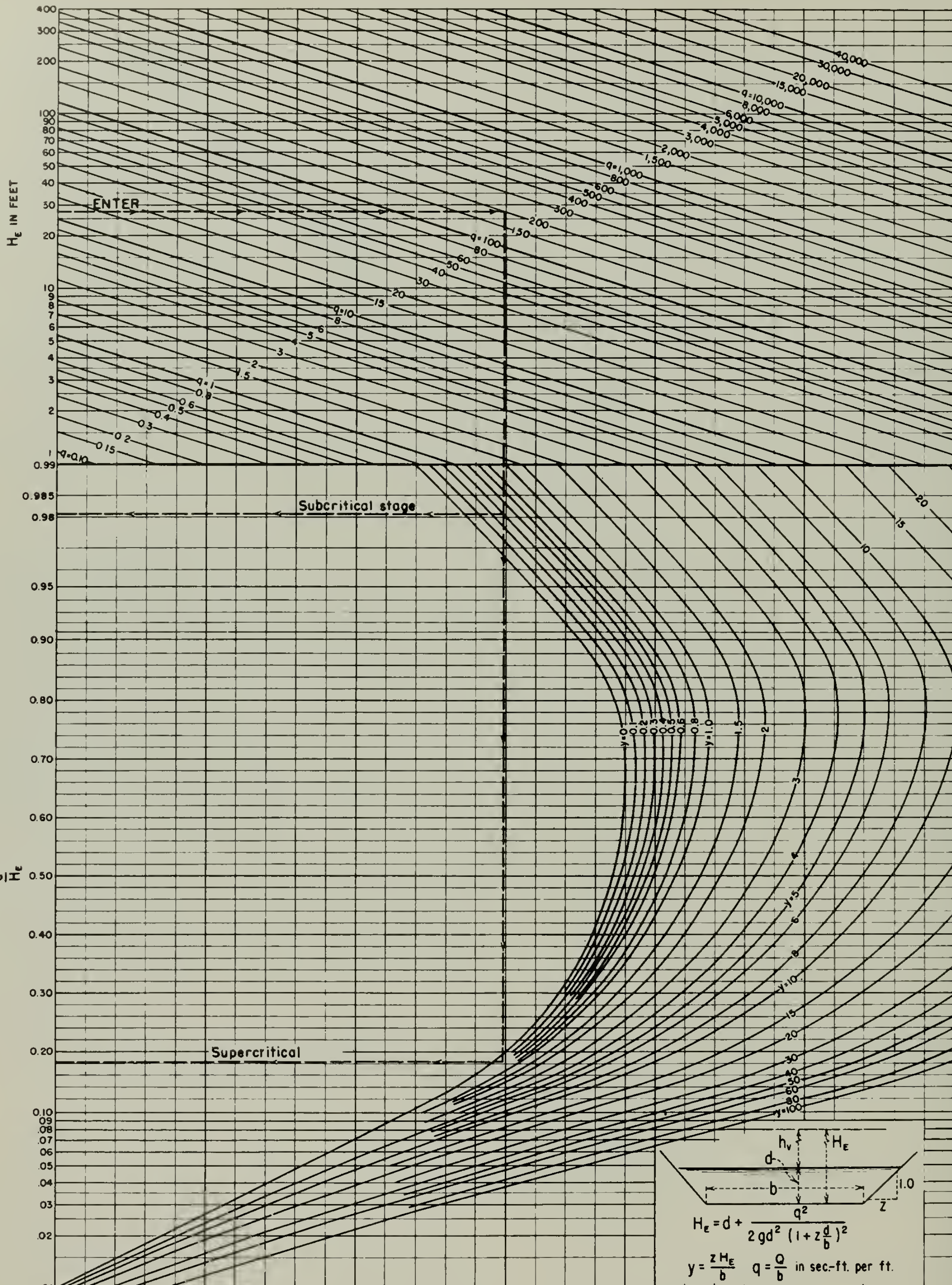


Figure B-3. Energy-depth curves for rectangular and trapezoidal channels.

Then equation (4) can be stated

$$H_E=d_c+\frac{d_{m_c}}{2} \tag{10}$$

From the foregoing, the following additional relations can be stated:

$$d_{m_c}=\frac{v_c^2}{g} \tag{11}$$

$$d_{m_c}=\frac{Q_c^2}{a^2g} \tag{12}$$

$$v_c=\sqrt{gd_{m_c}} \tag{13}$$

$$v_c=\sqrt{\frac{ag}{T}}=5.67\sqrt{\frac{a}{T}} \tag{14}$$

$$Q_c=a\sqrt{gd_{m_c}} \tag{15}$$

For rectangular sections, if q is the discharge per foot width of channel, the various critical flow formulae are:

$$H_{E_c}=\frac{3}{2}\,d_c \tag{16}$$

$$d_c=\frac{2}{3}\,H_{E_c} \tag{17}$$

$$d_c=\frac{v_c^2}{g} \tag{18}$$

$$d_c=\sqrt[3]{\frac{q_c^2}{g}} \tag{19}$$

$$d_c=\sqrt[3]{\frac{Q_c^2}{b^2g}} \tag{20}$$

$$v_c=\sqrt{gd_c} \tag{21}$$

$$v_c=\sqrt[3]{gq_c} \tag{22}$$

$$v_c=\sqrt[3]{\frac{gQ_c}{b}} \tag{23}$$

$$q_c=d_c^{3/2}\sqrt{g} \tag{24}$$

$$Q_c=5.67bd_c^{3/2} \tag{25}$$

$$Q_c=3.087bH_{E_c}^{3/2} \tag{26}$$

The critical depth for trapezoidal sections is given by the equation:

$$d_c=\frac{v_c^2}{g}-\frac{b}{2z}+\sqrt{\frac{v_c^4}{g^2}+\frac{b^2}{4z^2}} \tag{27}$$

where z = the ratio, horizontal to vertical, of the slope of the sides of the channel.

Similarly, for the trapezoidal section,

$$v_c=\sqrt{\left(\frac{b+zd_c}{b+2zd_c}\right)d_cg} \tag{28}$$

and

$$Q_c=d_c^{3/2}\sqrt{\frac{g(b+zd_c)^3}{b+2zd_c}} \tag{29}$$

The solutions of equations (25) and (29) are simplified by use of figure B-4.

A general equation for critical depth cannot be expressed for natural channels. However, a check for the existence of critical flow in these channels is discussed in part B of this appendix.

(c) *Manning Formula.*—The formula developed by Manning for flow in open channels is used in most of the hydraulic analyses discussed in this text. It is a special form of Chezy's formula; the complete development is contained in most textbooks on elementary fluid mechanics. The formula is written as follows:

$$v=\frac{1.486}{n}\,r^{2/3}s^{1/2} \tag{30}$$

or

$$Q=\frac{1.486}{n}\,ar^{2/3}s^{1/2} \tag{31}$$

where:

a = the cross section of flow area in square feet,
v = the velocity in feet per second,
n = a roughness coefficient,
r = the hydraulic radius = $\frac{\text{area } (a)}{\text{wetted perimeter } (p)}$, and
s = the slope of the energy gradient.

The value of the roughness coefficient, n, varies according to the physical roughness of the sides and bottom of the channel and is influenced by such factors as channel curvature, size and shape of cross section, alinement, and type and condition of the material forming the wetted perimeter.

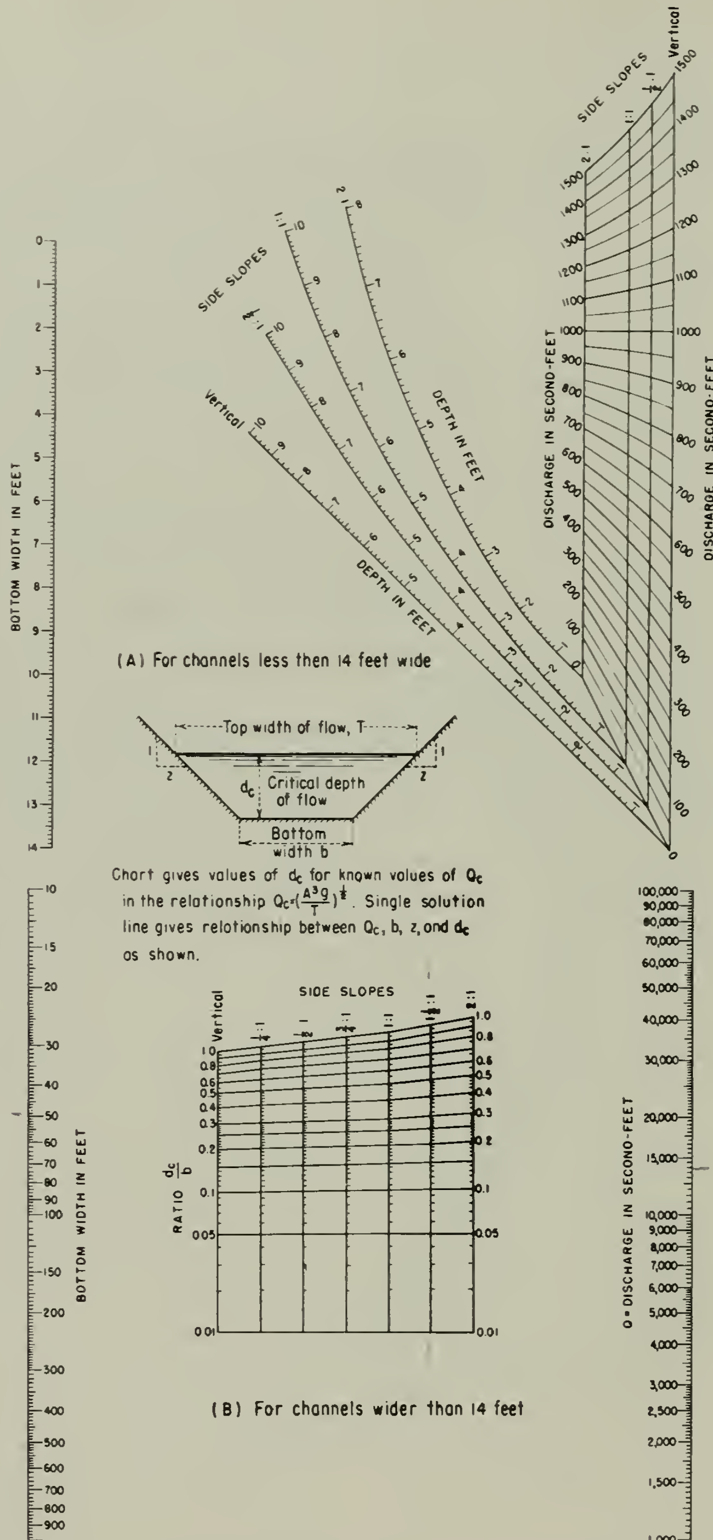


Figure B-4. **Critical depth in trapezoidal sections.**

After C. Freeman

Values of n commonly used in design of artificial channels are as follows:

Description of channel	Values of n		
	Minimum	Maximum	Average
Earth channels, straight and uniform	0.017	0.025	0.0225
Dredged earth channels	.025	0.033	.0275
Rock channels, straight and uniform	.025	.035	.033
Rock channels, jagged and irregular	.035	.045	.045
Concrete lined	.012	.018	.014
Neat cement lined	.010	.013	.....
Grouted rubble paving	.017	.030	.....
Corrugated metal	.023	.025	.024

The determination of n values for natural channels is discussed in part B of this appendix.

(d) *Bernoulli Theorem.*—The Bernoulli theorem, which is the principle of conservation of energy applied to open channel flow, may be stated: The absolute head at any section is equal to the absolute head at a section downstream plus intervening losses of head. Expressed in terms of equation (3), from figure B-1:

$$Z_2+d_2+h_{v_2}=Z_1+d_1+h_{v_1}+h_L \tag{32}$$

where h_L represents all losses in head between section 2 (subscript 2) and section 1 (subscript 1). Such head losses will consist largely of friction loss, but may include minor other losses such as those due to eddy, transition, obstruction, impact, etc.

When the discharge at a given cross section of a channel is constant with respect to time, the flow is steady. If steady flow occurs at all sections in a reach, the flow is continuous and

$$Q=a_1v_1=a_2v_2 \tag{33}$$

Equation (33) is termed the equation of continuity. Equations (32) and (33), solved simultaneously, are the basic formulas used in solving problems of flow in open channels.

(e) *Hydraulic and Energy Gradients.*—The hydraulic gradient in open channel flow is the water surface. The energy gradient is above the hydraulic gradient a distance equal to the velocity head. The fall of the energy gradient for a given length of channel represents the loss of energy, either from friction or from friction and other influences. The relationship of the energy gradient to the hydraulic gradient reflects not only the loss of energy, but also the conversion between potential and kinetic energy. For uniform flow the gradients are parallel and the slope of the water surface represents the friction loss gradient. In accelerated flow the hydraulic gradient is steeper than the energy gradient, indicating a progressive conversion from potential to kinetic energy. In retarded flow the energy gradient is steeper than the hydraulic gradient, indicating a conversion from kinetic to potential energy. The Bernoulli theorem defines the progressive relationships of these energy gradients.

For a given reach of channel ΔL, the average slope of the energy gradient is $\frac{\Delta h_L}{\Delta L}$, where Δh_L is the cumulative losses through the reach. If these losses are solely from friction, Δh_L will become Δh_f and

$$\Delta h_f=\left(\frac{s_2+s_1}{2}\right)\Delta L \tag{34}$$

Expressed in terms of the hydraulic properties at each end of the reach and of the roughness coefficient,

$$\Delta h_f=\frac{n^2}{4.41}\left[\left(\frac{v_2}{r_2^{2/3}}\right)^2+\left(\frac{v_1}{r_1^{2/3}}\right)^2\right]\Delta L \tag{35}$$

If the average friction slope, s_f, is equal to $\frac{s_2+s_1}{2}=\frac{\Delta h_f}{\Delta L}$, and s_b is the slope of the channel floor, by substituting $s_b\Delta L$ for Z_2-Z_1, and H_E for $(d+h_v)$, equation (32) may be written:

$$\Delta L=\frac{H_{E_1}-H_{E_2}}{s_b-s_f} \tag{36}$$

(f) *Chart for Approximating Friction Losses in Chutes.*—Figure B-5 is a nomograph from which approximate friction losses in a channel can be evaluated. To generalize the chart so that it can be applied for differing channel conditions, several approximations are made. First, the depth of flow in the channel is assumed equal to the hydraulic radius; the results will therefore be most applicable to wide, shallow channels. Furthermore, the increase in velocity head is assumed to vary proportionally along the length of the channel. Thus, the data given in the chart are not exact and are intended to serve only as a guide in estimating channel losses.

The chart plots the solution of the equation

$s=\frac{dh_f}{dx}$, integrated between the limits from zero to L, or

$$h_f=\int_0^L s\,dx,$$

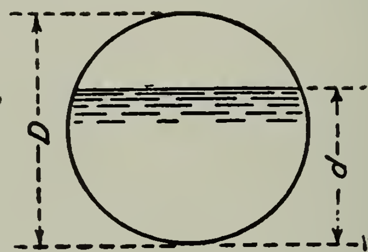
where, from the Manning equation,

$$s=\frac{v^2}{\left(\frac{1.486}{n}\right)^2 r^{4/3}}$$

B-3. *Flow in Closed Conduits.*—(a) *Part Full Flow in Conduits.*—The hydraulics of part full flow in closed conduits is similar to that in open channels, and open channel flow formulas are applicable. Hydraulic properties for different flow depths in circular and horseshoe conduits are tabulated in tables B-2 through B-5 to facilitate hydraulic computations for these sections.

TABLE B-2.—*Velocity head and discharge at critical depths and static pressures in circular conduits partly full*

D=Diameter of pipe.
d=Depth of flow.
h_{v_c}=Velocity head for a critical depth of d.
Q_c=Discharge when the critical depth is d.
P=Pressure on cross section of water prism in cubic units of water. To get P in pounds, when d and D are in feet, multiply by 62.5.



$\frac{d}{D}$	$\frac{h_{v_c}}{D}$	$\frac{Q_c}{D^{5/2}}$	$\frac{P}{D^3}$	$\frac{d}{D}$	$\frac{h_{v_c}}{D}$	$\frac{Q_c}{D^{5/2}}$	$\frac{P}{D^3}$	$\frac{d}{D}$	$\frac{h_{v_c}}{D}$	$\frac{Q_c}{D^{5/2}}$	$\frac{P}{D^3}$
1	2	3	4	1	2	3	4	1	2	3	4
0.01	0.0033	0.0006	0.0000	0.34	0.1243	0.6657	0.0332	0.67	0.2974	2.4464	0.1644
.02	.0067	.0025	.0000	.35	.1284	.7040	.0356	.68	.3048	2.5182	.1700
.03	.0101	.0055	.0001	.36	.1326	.7433	.0381	.69	.3125	2.5912	.1758
.04	.0134	.0098	.0002	.37	.1368	.7836	.0407	.70	.3204	2.6656	.1816
.05	.0168	.0153	.0003	.38	.1411	.8249	.0434	.71	.3286	2.7414	.1875
.06	.0203	.0220	.0005	.39	.1454	.8671	.0462	.72	.3371	2.8188	.1935
.07	.0237	.0298	.0007	.40	.1497	.9103	.0491	.73	.3459	2.8977	.1996
.08	.0271	.0389	.0010	.41	.1541	.9545	.0520	.74	.3552	2.9783	.2058
.09	.0306	.0491	.0013	.42	.1586	.9996	.0551	.75	.3648	3.0607	.2121
.10	.0341	.0605	.0017	.43	.1631	1.0458	.0583	.76	.3749	3.1450	.2185
.11	.0376	.0731	.0021	.44	.1676	1.0929	.0616	.77	.3855	3.2314	.2249
.12	.0411	.0868	.0026	.45	.1723	1.1410	.0650	.78	.3967	3.3200	.2314
.13	.0446	.1016	.0032	.46	.1769	1.1899	.0684	.79	.4085	3.4112	.2380
.14	.0482	.1176	.0038	.47	.1817	1.2399	.0720	.80	.4210	3.5050	.2447
.15	.0517	.1347	.0045	.48	.1865	1.2908	.0757	.81	.4343	3.6019	.2515
.16	.0553	.1530	.0053	.49	.1914	1.3427	.0795	.82	.4485	3.7021	.2584
.17	.0589	.1724	.0061	.50	.1964	1.3955	.0833	.83	.4638	3.8061	.2653
.18	.0626	.1928	.0070	.51	.2014	1.4493	.0873	.84	.4803	3.9144	.2723
.19	.0662	.2144	.0080	.52	.2065	1.5041	.0914	.85	.4982	4.0276	.2794
.20	.0699	.2371	.0091	.53	.2117	1.5598	.0956	.86	.5177	4.1465	.2865
.21	.0736	.2609	.0103	.54	.2170	1.6164	.0998	.87	.5392	4.2721	.2938
.22	.0773	.2857	.0115	.55	.2224	1.6735	.1042	.88	.5632	4.4056	.3011
.23	.0811	.3116	.0128	.56	.2279	1.7327	.1087	.89	.5900	4.5486	.3084
.24	.0848	.3386	.0143	.57	.2335	1.7923	.1133	.90	.6204	4.7033	.3158
.25	.0887	.3667	.0157	.58	.2393	1.8530	.1179	.91	.6555	4.8725	.3233
.26	.0925	.3957	.0173	.59	.2451	1.9146	.1227	.92	.6966	5.0603	.3308
.27	.0963	.4259	.0190	.60	.2511	1.9773	.1276	.93	.7459	5.2726	.3384
.28	.1002	.4571	.0207	.61	.2572	2.0409	.1326	.94	.8065	5.5183	.3460
.29	.1042	.4893	.0226	.62	.2635	2.1057	.1376	.95	.8841	5.8118	.3537
.30	.1081	.5225	.0255	.63	.2699	2.1716	.1428	.96	.9885	6.1787	.3615
.31	.1121	.5568	.0266	.64	.2765	2.2386	.1481	.97	1.1410	6.6692	.3692
.32	.1161	.5921	.0287	.65	.2833	2.3067	.1534	.98	1.3958	7.4063	.3770
.33	.1202	.6284	.0309	.66	.2902	2.3760	.1589	.99	1.9700	8.8263	.3848
								1.00	------	------	.3927

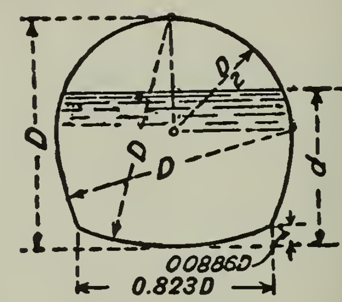
TABLE B-3. *Uniform flow in circular sections flowing partly full*

d = Depth of flow
D = Diameter of pipe
A = Area of flow
r = Hydraulic radius
Q = Discharge in second-feet by Manning's formula
n = Manning's coefficient
s = Slope of the channel bottom and of the water surface

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{r}{D}$	$\frac{Qn}{D^{8/3}s^{1/2}}$	$\frac{Qn}{d^{8/3}s^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{r}{D}$	$\frac{Qn}{D^{8/3}s^{1/2}}$	$\frac{Qn}{d^{8/3}s^{1/2}}$
1	2	3	4	5	1	2	3	4	5
0.01	0.0013	0.0066	0.00007	15.04	0.51	0.4027	0.2531	0.239	1.442
.02	.0037	.0132	.00031	10.57	.52	.4127	.2562	.247	1.415
.03	.0069	.0197	.00074	8.56	.53	.4227	.2592	.255	1.388
.04	.0105	.0262	.00138	7.38	.54	.4327	.2621	.263	1.362
.05	.0147	.0325	.00222	6.55	.55	.4426	.2649	.271	1.336
.06	.0192	.0389	.00328	5.95	.56	.4526	.2676	.279	1.311
.07	.0242	.0451	.00455	5.47	.57	.4625	.2703	.287	1.286
.08	.0294	.0513	.00604	5.09	.58	.4724	.2728	.295	1.262
.09	.0350	.0575	.00775	4.76	.59	.4822	.2753	.303	1.238
.10	.0409	.0635	.00967	4.49	.60	.4920	.2776	.311	1.215
.11	.0470	.0695	.01181	4.25	.61	.5018	.2799	.319	1.192
.12	.0534	.0755	.01417	4.04	.62	.5115	.2821	.327	1.170
.13	.0600	.0813	.01674	3.86	.63	.5212	.2842	.335	1.148
.14	.0668	.0871	.01952	3.69	.64	.5308	.2862	.343	1.126
.15	.0739	.0929	.0225	3.54	.65	.5404	.2882	.350	1.105
.16	.0811	.0985	.0257	3.41	.66	.5499	.2900	.358	1.084
.17	.0885	.1042	.0291	3.28	.67	.5594	.2917	.366	1.064
.18	.0961	.1097	.0327	3.17	.68	.5687	.2933	.373	1.044
.19	.1039	.1152	.0365	3.06	.69	.5780	.2948	.380	1.024
.20	.1118	.1206	.0406	2.96	.70	.5872	.2962	.388	1.004
.21	.1199	.1259	.0448	2.87	.71	.5964	.2975	.395	0.985
.22	.1281	.1312	.0492	2.79	.72	.6054	.2987	.402	.965
.23	.1365	.1364	.0537	2.71	.73	.6143	.2998	.409	.947
.24	.1449	.1416	.0585	2.63	.74	.6231	.3008	.416	.928
.25	.1535	.1466	.0634	2.56	.75	.6319	.3017	.422	.910
.26	.1623	.1516	.0686	2.49	.76	.6405	.3024	.429	.891
.27	.1711	.1566	.0739	2.42	.77	.6489	.3031	.435	.873
.28	.1800	.1614	.0793	2.36	.78	.6573	.3036	.441	.856
.29	.1890	.1662	.0849	2.30	.79	.6655	.3039	.447	.838
.30	.1982	.1709	.0907	2.25	.80	.6736	.3042	.453	.821
.31	.2074	.1756	.0966	2.20	.81	.6815	.3043	.458	.804
.32	.2167	.1802	.1027	2.14	.82	.6893	.3043	.463	.787
.33	.2260	.1847	.1089	2.09	.83	.6969	.3041	.468	.770
.34	.2355	.1891	.1153	2.05	.84	.7043	.3038	.473	.753
.35	.2450	.1935	.1218	2.00	.85	.7115	.3033	.477	.736
.36	.2546	.1978	.1284	1.958	.86	.7186	.3026	.481	.720
.37	.2642	.2020	.1351	1.915	.87	.7254	.3018	.485	.703
.38	.2739	.2062	.1420	1.875	.88	.7320	.3007	.488	.687
.39	.2836	.2102	.1490	1.835	.89	.7384	.2995	.491	.670
.40	.2934	.2142	.1561	1.797	.90	.7445	.2980	.494	.654
.41	.3032	.2182	.1633	1.760	.91	.7504	.2963	.496	.637
.42	.3130	.2220	.1705	1.724	.92	.7560	.2944	.497	.621
.43	.3229	.2258	.1779	1.689	.93	.7612	.2921	.498	.604
.44	.3328	.2295	.1854	1.655	.94	.7662	.2895	.498	.588
.45	.3428	.2331	.1929	1.622	.95	.7707	.2865	.498	.571
.46	.3527	.2366	.201	1.590	.96	.7749	.2829	.496	.553
.47	.3627	.2401	.208	1.559	.97	.7785	.2787	.494	.535
.48	.3727	.2435	.216	1.530	.98	.7817	.2735	.489	.517
.49	.3827	.2468	.224	1.500	.99	.7841	.2666	.483	.496
.50	.3927	.2500	.232	1.471	1.00	.7854	.2500	.463	.463

TABLE B-4.—*Velocity head and discharge at critical depths and static pressures in horseshoe conduits partly full*

D = Diameter of horseshoe.
d = Depth of flow.
h_{v_c} = Velocity head for a critical depth of d.
Q_c = Discharge when the critical depth is d.
P = Pressure on cross section of water prism in cubic units of water. To get P in pounds, when d and D are in feet, multiply by 62.5.



$\frac{d}{D}$	$\frac{h_{v_c}}{D}$	$\frac{Q_c}{D^{5/2}}$	$\frac{P}{D^3}$	$\frac{d}{D}$	$\frac{h_{v_c}}{D}$	$\frac{Q_c}{D^{5/2}}$	$\frac{P}{D^3}$	$\frac{d}{D}$	$\frac{h_{v_c}}{D}$	$\frac{Q_c}{D^{5/2}}$	$\frac{P}{D^3}$
1	2	3	4	1	2	3	4	1	2	3	4
0.01	0.0033	0.0009	0.0000	0.35	0.1472	0.8854	0.0449	0.69	0.3362	2.8922	0.1999
.02	.0067	.0035	.0000	.36	.1518	.9296	.0478	.70	.3413	2.9702	.2062
.03	.0100	.0079	.0001	.37	.1563	.9746	.0508	.71	.3528	3.0499	.2125
.04	.0134	.0139	.0002	.38	.1609	1.0205	.0540	.72	.3615	3.1311	.2190
.05	.0168	.0217	.0004	.39	.1655	1.0673	.0572	.73	.3707	3.2140	.2255
.06	.0201	.0312	.0007	.40	.1702	1.1148	.0605	.74	.3802	3.2987	.2321
.07	.0235	.0425	.0010	.41	.1749	1.1633	.0639	.75	.3902	3.3853	.2385
.08	.0269	.0554	.0014	.42	.1795	1.2125	.0675	.76	.4006	3.4740	.2457
.09	.0305	.0703	.0018	.43	.1843	1.2626	.0711	.77	.4116	3.5650	.2525
.10	.0351	.0879	.0024	.44	.1890	1.3135	.0748	.78	.4232	3.6584	.2595
.11	.0397	.1069	.0030	.45	.1938	1.3652	.0786	.79	.4354	3.7544	.2666
.12	.0443	.1272	.0037	.46	.1986	1.4178	.0825	.80	.4484	3.8534	.2737
.13	.0489	.1487	.0045	.47	.2035	1.4712	.0865	.81	.4623	3.9557	.2809
.14	.0534	.1714	.0054	.48	.2084	1.5253	.0907	.82	.4771	4.0616	.2882
.15	.0579	.1953	.0063	.49	.2133	1.5803	.0949	.83	.4930	4.1716	.2956
.16	.0624	.2203	.0074	.50	.2183	1.6361	.0992	.84	.5102	4.2863	.3030
17	.0669	.2465	.0085	.51	.2234	1.6928	.1036	.85	.5289	4.4063	.3105
.18	.0714	.2736	.0098	.52	.2285	1.7505	.1081	.86	.5494	4.5325	.3181
.19	.0758	.3019	.0111	.53	.2337	1.8092	.1127	.87	.5719	4.6660	.3258
.20	.0803	.3312	.0125	.54	.2391	1.8688	.1174	.88	.5969	4.8080	.3335
.21	.0847	.3615	.0140	.55	.2445	1.9294	.1223	.89	.6251	4.9605	.3413
.22	.0891	.3928	.0156	.56	.2500	1.9911	.1272	.90	.6570	5.1256	.3492
.23	.0936	.4251	.0173	.57	.2557	2.0537	.1322	.91	.6939	5.3065	.3572
.24	.0980	.4583	.0191	.58	.2615	2.1174	.1373	.92	.7371	5.5077	.3653
.25	.1024	.4926	.0210	.59	.2674	2.1821	.1425	.93	.7889	5.7354	.3733
.26	.1069	.5277	.0229	.60	.2735	2.2479	.1478	.94	.8528	5.9996	.3813
.27	.1113	.5638	.0250	.61	.2797	2.3148	.1532	.95	.9345	6.3157	.3894
.28	.1158	.6009	0271	.62	.2861	2.3828	.1587	.96	1.0446	6.7114	.3976
.29	.1202	.6389	.0294	.63	.2926	2.4519	.1643	.97	1.2053	7.2417	.4058
.30	.1247	.6777	.0317	.64	.2994	2.5221	.1700	.98	1.4742	8.0892	4140
.31	.1292	.7175	.0342	.65	.3063	2.5936	.1758	.99	2.0804	9.5780	.4223
.32	.1337	.7582	.0367	.66	.3134	2.6663	.1817	1.00	------------	------------	.4306
.33	.1382	.7997	.0393	.67	.3208	2.7402	.1877				
.34	.1427	.8421	.0421	.68	.3283	2.8155	.1937				

TABLE B-5.—*Uniform flow in horseshoe sections flowing partly full*

d = Depth of flow.
D = Diameter.
A = Area of flow.
r = Hydraulic radius.

Q = Discharge in second feet by Manning's formula.
n = Manning's coefficient.
s = Slope of the channel bottom and of the water surface.

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{r}{D}$	$\frac{Qn}{D^{8/3}s^{1/2}}$	$\frac{Qn}{d^{8/3}s^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{r}{D}$	$\frac{Qn}{D^{8/3}s^{1/2}}$	$\frac{Qn}{d^{8/3}s^{1/2}}$
1	2	3	4	5	1	2	3	4	5
0.01	0.0019	0.0066	0.00010	21.40	0.51	0.4466	0.2602	0.2705	1.629
.02	.0053	.0132	.00044	14.93	.52	.4566	.2630	.2785	1.593
.03	.0097	.0198	.00105	12.14	.53	.4666	.2657	.2866	1.558
.04	.0150	.0264	.00198	10.56	.54	.4766	.2683	.2946	1.524
.05	.0209	.0329	.00319	9.40	.55	.4865	.2707	.303	1.490
.06	.0275	.0394	.00473	8.58	.56	.4965	.2733	.311	1.458
.07	.0346	.0459	.00659	7.92	.57	.5064	.2757	.319	1.427
.08	.0421	.0524	.00876	7.37	.58	.5163	.2781	.327	1.397
.09	.0502	.0590	.01131	6.95	.59	.5261	.2804	.335	1.368
.10	.0585	.0670	.01434	6.66	.60	.5359	.2824	.343	1.339
.11	.0670	.0748	.01768	6.36	.61	.5457	.2844	.351	1.310
.12	.0753	.0823	.02117	6.04	.62	.5555	.2864	.359	1.283
.13	.0839	.0895	.02495	5.75	.63	.5651	.2884	.367	1.257
.14	.0925	.0964	.02890	5.47	.64	.5748	.2902	.374	1.231
.15	.1012	.1031	.0331	5.21	.65	.5843	.2920	.382	1.206
.16	.1100	.1097	.0375	4.96	.66	.5938	.2937	.390	1.181
.17	.1188	.1161	.0420	4.74	.67	.6033	.2953	.398	1.157
.18	.1277	.1222	.0467	4.52	.68	.6126	.2967	.405	1.133
.19	.1367	.1282	.0516	4.33	.69	.6219	.2981	.412	1.109
.20	.1457	.1341	.0567	4.15	.70	.6312	.2994	.420	1.087
.21	.1549	.1398	.0620	3.98	.71	.6403	.3006	.427	1.064
.22	.1640	.1454	.0674	3.82	.72	.6493	.3018	.434	1.042
.23	.1733	.1508	.0730	3.68	.73	.6582	.3028	.441	1.021
.24	.1825	.1560	.0786	3.53	.74	.6671	.3036	.448	1.000
.25	.1919	.1611	.0844	3.40	.75	.6758	.3044	.454	0.979
.26	.2013	.1662	.0904	3.28	.76	.6844	.3050	.461	.958
.27	.2107	.1710	.0965	3.17	.77	.6929	.3055	.467	.938
.28	.2202	.1758	.1027	3.06	.78	.7012	.3060	.473	.918
.29	.2297	.1804	.1090	2.96	.79	.7094	.3064	.479	.898
.30	.2393	.1850	.1155	2.86	.80	.7175	.3067	.485	.879
.31	.2489	.1895	.1220	2.77	.81	.7254	.3067	.490	.860
.32	.2586	.1938	.1287	2.69	.82	.7332	.3066	.495	.841
.33	.2683	.1981	.1355	2.61	.83	.7408	.3064	.500	.822
.34	.2780	.2023	.1424	2.53	.84	.7482	.3061	.505	.804
.35	.2878	.2063	.1493	2.45	.85	.7554	.3056	.509	.786
.36	.2975	.2103	.1563	2.38	.86	.7625	.3050	.513	.768
.37	.3074	.2142	.1635	2.32	.87	.7693	.3042	.517	.750
.38	.3172	.2181	.1708	2.25	.88	.7759	.3032	.520	.732
.39	.3271	.2217	.1781	2.19	.89	.7823	.3020	.523	.714
.40	.3370	.2252	.1854	2.13	.90	.7884	.3005	.526	.696
.41	.3469	.2287	.1928	2.08	.91	.7943	.2988	.528	.678
.42	.3568	.2322	.2003	2.02	.92	.7999	.2969	.529	.661
.43	.3667	.2356	.2079	1.973	.93	.8052	.2947	.530	.643
.44	.3767	.2390	.2156	1.925	.94	.8101	.2922	.530	.625
.45	.3867	.2422	.2233	1.878	.95	.8146	.2893	.529	.607
.46	.3966	.2454	.2310	1.832	.96	.8188	.2858	.528	.589
.47	.4066	.2484	.2388	1.788	.97	.8224	.2816	.525	.569
.48	.4166	.2514	.2466	1.746	.98	.8256	.2766	.521	.550
.49	.4266	.2544	.2545	1.705	.99	.8280	.2696	.513	.527
.50	.4366	.2574	.2625	1.667	1.00	.8293	.2538	.494	.494

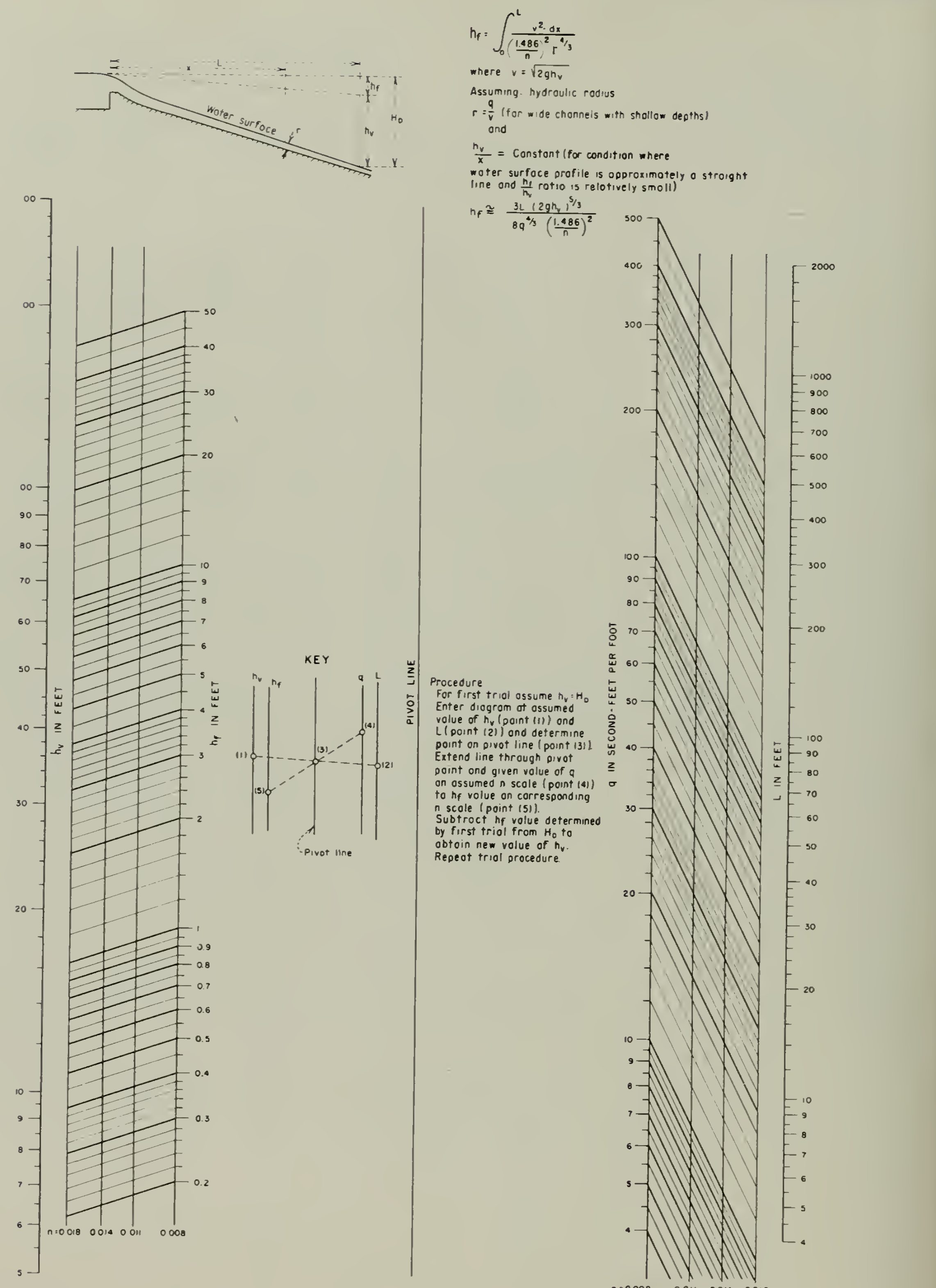


Figure B-5. Approximate losses in chutes for various values of water surface drop and channel length.

After O. Pfofstetter

Tables B–2 and B–4 give data for determining critical depths, critical velocities, and hydrostatic pressures of the water prism cross section for various discharges and conduit diameters. If the area at critical flow, a_c, is represented as k_1D^2 and the top width of the water prism, T, for critical flow is equal to k_2D, equation (7) can be written:

$$\frac{Q_c^2}{g}=\frac{(k_1D^2)^3}{k_2D}, \text{ or } Q_c=k_3D^{5/2} \qquad (37)$$

Values of k_3, for various flow depths, are tabulated in column 3. The hydrostatic pressure, P, of the water prism cross section is $wa\bar{y}$, where $\bar{y}$ is the depth from the water surface to the center of gravity of the cross section. If $a_c=k_1D^2$ and $\bar{y}=k_4D$, then

$$P=k_5D^3 \qquad (38)$$

Values of k_5, for various flow depths, are tabulated in column 4. Column 2 gives the values of h_{v_c} in relation to the conduit diameter, for various flow depths.

The use of tables B–2 and B–4 can be illustrated by an example. Suppose that it is desired to find the critical depth for 650 second-feet flowing free in a 9-foot-diameter circular conduit. For this case, $\frac{Q}{d^{5/2}}=\frac{650}{243}=2.675$. From column 3, by interpolation, the corresponding value in column 1 is $\frac{d}{D}=0.701$. The critical depth is then $9\times0.701=6.31$ feet. The critical velocity from column 2 is $h_{v_c}=0.3212\times9=2.89$ feet, which gives a critical velocity of 13.6 feet per second.

Column 4 gives hydrostatic pressure upon the cross section of the water prism. The tabular value multiplied by D^3 gives pressure in cubic units of water. The pressure in pounds is 62.5 times the tabular value. For this example, $P=62.5\times 9^3\times0.1822=8{,}300$ pounds.

Tables B–3 and B–5 give areas and hydraulic radii for partially full conduits and coefficients which can be applied in the solution of the Manning equation. If $A=k_6\frac{\pi D^2}{4}$ and $r=k_7D$, Manning's equation can be written

$$Q=\frac{1.486}{n}\left(k_6\frac{\pi D^2}{4}\right)(k_7D)^{2/3}s^{1/2},$$

or

$$\frac{Qn}{D^{8/3}s^{1/2}}=k_6\frac{1.486\pi}{4}(k_7)^{2/3}=k_8 \qquad (39)$$

Values of k_8, for various flow depths, are tabulated in column 4. If $D=k_9d$, equation (39) can be written:

$$\frac{Qn}{d^{8/3}s^{1/2}}=\frac{1.486\pi}{4}k_6(k_7)^{2/3}(k_9)^{8/3}=k_{10} \qquad (40)$$

Values of k_{10}, for various flow depths, are tabulated in column 5.

(b) *Pressure Flow in Conduits.*—Since factors affecting head losses in conduits are independent of pressure, the same laws apply to flow in both closed conduits and open channels, and the formulas for each take the same general form. Thus, the equation of continuity, equation (33), $Q=a_1v_1=a_2v_2$, also applies to pressure flow in conduits.

A mass of water as such does not have pressure energy. Pressure energy is acquired by contact with other masses and is, therefore, transmitted to or through the mass under consideration. The pressure head $\frac{p}{w}$ (where p is the pressure intensity in pounds per square foot and w is unit weight in pounds per cubic foot), like velocity and elevation heads, also expresses energy. Thus for pressure pipe flow, the Bernoulli equation for flow in open channels, equation (3), can be written:

$$H_A=\frac{p}{w}+Z+\frac{v^2}{2g} \qquad (41)$$

The Bernoulli theorem for flow in a reach of pressure pipe (as shown on fig. B–6) is:

$$\frac{p_1}{w}+Z_1+h_{v_1}=\frac{p_2}{w}+Z_2+h_{v_2}+\Delta h_L \qquad (42)$$

where Δh_L represents the head losses within the reach from all causes. If H_T is the total head and v is the velocity at the outlet, Bernoulli's equation for the entire pipe is:

$$H_T=\Sigma(\Delta h_L)+h_v \qquad (43)$$

As in open channel flow, the Bernoulli theorem and the continuity equation are the basic formulas used in solving problems of pressure conduit flow.

(c) *Energy and Pressure Gradients.*—If piezometer stand pipes were to be inserted at various points along the length of a conduit flowing under pressure, as illustrated on figure B–6, water would rise in each pipe to a level equal to the pressure head in the conduit at those points. The level may be equal to the pipe grade or be above or below it; that is, the pressure at that point may be equal to, greater than, or less than the local atmospheric pressure. The height to which the water would rise in a piezometer is termed the pressure gradient. The energy gradient is above the pressure gradient a distance equal to the velocity head. The fall of the energy gradient for a given length of pipe represents the loss of energy, either from friction or from friction and other influences. The relationship of the energy gradient to the pressure gradient reflects the variations between kinetic energy and pressure head.

(d) *Friction Losses.*—Many empirical formulas have been developed for evaluating the flow of fluids in pipes. Those in most common use are the Manning equation, which, written in terms of the pipe length and diameter (in feet), is:

$$h_f = 185 n^2 \frac{L}{D^{4/3}} \frac{v^2}{2g} \tag{44}$$

and the Darcy-Weisbach equation, which, written in similar terms, is:

$$h_f = \frac{fL}{D} \frac{v^2}{2g} \tag{45}$$

The Manning equation assumes that the energy loss depends only on the velocity, the dimensions of the conduit, and the magnitude of wall roughness as defined by the friction coefficient n. The n value is related to the physical roughness of the conduit wall and is independent of the size of the pipe or of the density and viscosity of the water.

The Darcy-Weisbach equation assumes the loss to be related to the velocity, the dimensions of the conduit, and the friction factor f. The factor f is a dimensionless variable based on the viscosity and density of the fluid and on the roughness of the conduit walls as it relates to the size of the conduit.

Data and criteria for determining f values for large pipe are given in a Bureau of Reclamation engineering monograph [6]. A value of the Manning coefficient, n, corresponding to the Darcy coefficient, f, can be obtained by combining equations (44) and (45). Thus:

$$n = \left(\frac{fD^{1/3}}{185}\right)^{1/2} \text{ or } f = \frac{185 n^2}{D^{1/3}} \tag{46}$$

The relationship between these coefficients is shown in nomograph form on figure B–7.

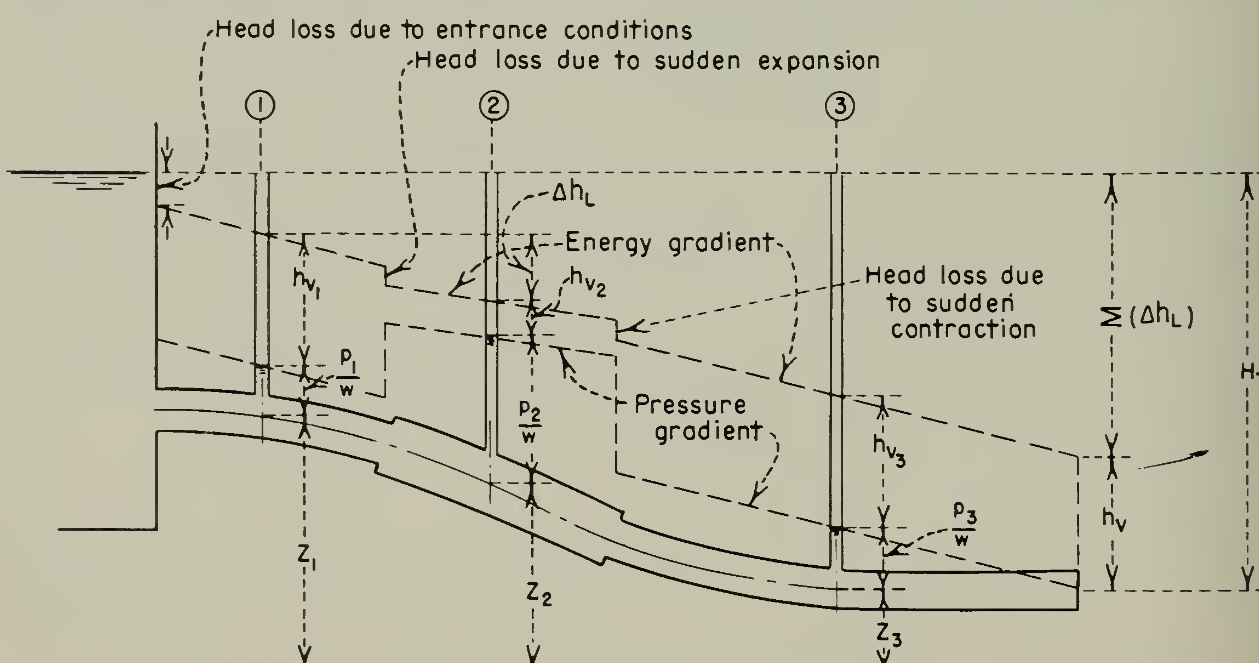


Figure B–6. Characteristics of pressure flow in conduits.

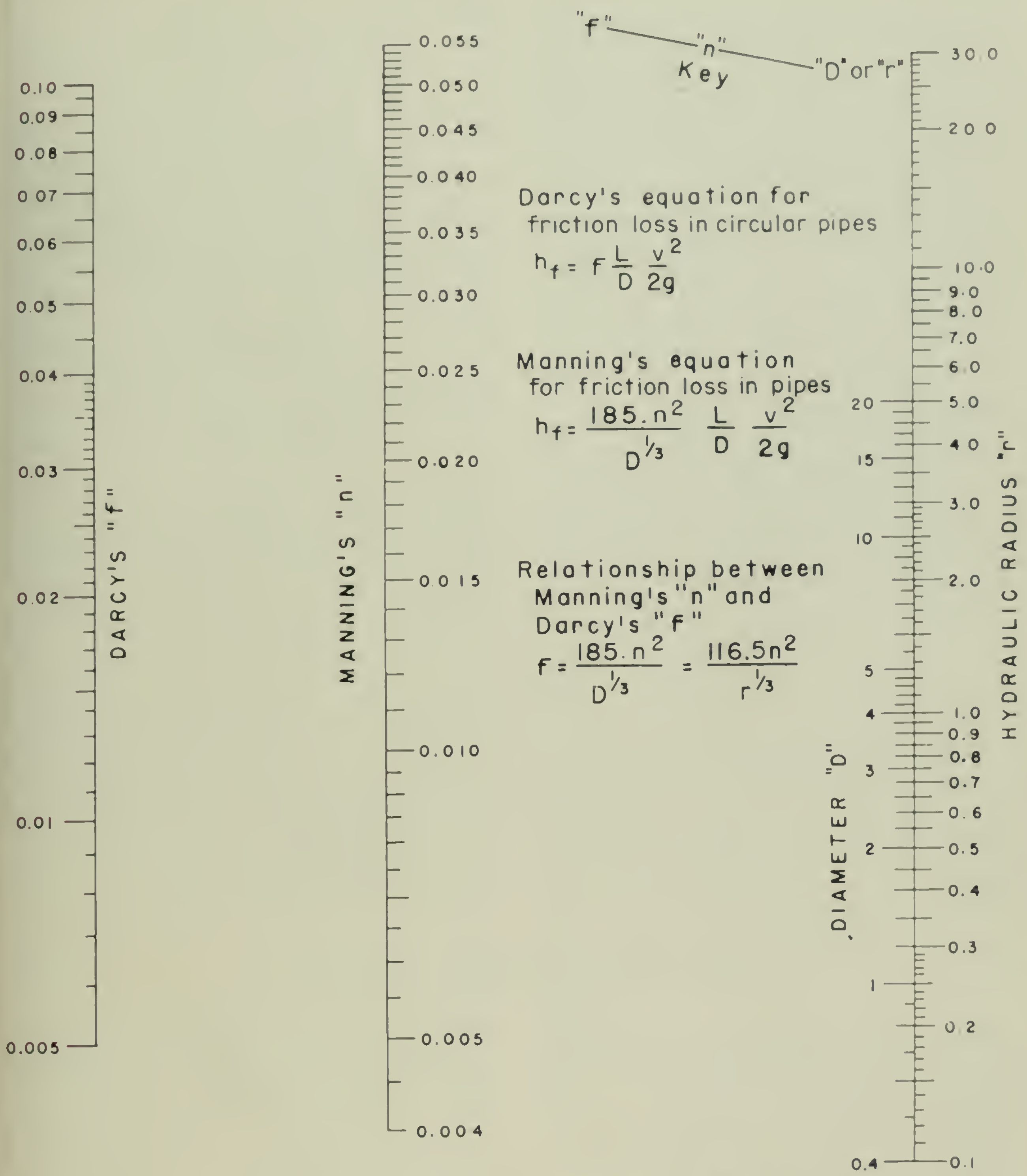


Figure B-7. Relationship between Darcy's *f* and Manning's *n* for flow in pipes.

(e) *Design Charts for Flow in Culverts.*—Figures B–8 through B–13 are nomographs prepared by the Bureau of Public Roads, presenting data which can be used for determining flow in circular and box culvert spillways. Figures B–8 and B–9 give discharges for pipes for conditions where the control is at the inlet; these charts are based on experimental data. Figures B–10 and B–11 give discharges for pipes flowing full, and represent the solution of equation (33) of chapter VIII. Similarly, figure B–12 gives discharges through box culverts with entrance control representing the solution of equation (35) of chapter VIII, with the value of the coefficient C_d determined experimentally. Figure B–13 gives discharges for box culverts flowing full, representing the solution of equation (38) of chapter VIII.

B–4. ***Hydraulic Jump.***—The hydraulic jump is an abrupt rise in water surface which may occur in an open channel when water flowing at high velocity is retarded. The formula for the hydraulic jump is obtained by equating the unbalanced forces acting to retard the mass of flow to the rate of change of the momentum of flow. The general formula for this relationship is:

$$v_1^2=g\,\frac{a_2\bar{y}_2-a_1\bar{y}_1}{a_1\left(1-\frac{a_1}{a_2}\right)} \tag{47}$$

where:

v_1=the velocity before the jump,

a_1 and a_2=the areas before and after the jump, respectively, and

$\bar{y}_1$ and $\bar{y}_2$=the corresponding depths from the water surface to the center of gravity of the cross section.

The general formula expressed in terms of discharge is:

$$Q^2=g\,\frac{a_2\bar{y}_2-a_1\bar{y}_1}{\frac{1}{a_1}-\frac{1}{a_2}} \tag{48}$$

or:

$$\frac{Q^2}{ga_1}+a_1\bar{y}_1=\frac{Q^2}{ga_2}+a_2\bar{y}_2 \tag{49}$$

For a rectangular channel, equation (47) can be reduced to $v_1^2=\frac{gd_2}{2d_1}(d_2+d_1)$, where d_1 and d_2 are the flow depths before and after the jump, respectively. Solving for d_2:

$$d_2=-\frac{d_1}{2}+\sqrt{\frac{2v_1^2d_1}{g}+\frac{d_1^2}{4}} \tag{50}$$

Similarly, expressing d_1 in terms of d_2 and v_2:

$$d_1=-\frac{d_2}{2}+\sqrt{\frac{2v_2^2d_2}{g}+\frac{d_2^2}{4}} \tag{51}$$

A graphic solution of equation (50) is shown on figure B–16.

If the Froude number $F_1=\frac{v_1}{\sqrt{gd_1}}$ is substituted in the equation (50):

$$\frac{d_2}{d_1}=\frac{1}{2}\left(\sqrt{8F_1^2+1}-1\right) \tag{52}$$

Figure B–14 shows a graphical representation of the characteristics of the hydraulic jump. Figure B–15 shows the hydraulic properties of the jump in relation to the Froude number, as determined from experimental data [7]. Data are for jumps on a flat floor with no chute blocks, baffle piers or end sills. Ordinarily, the jump length can be shortened by incorporation of such devices in the designs of a specific stilling basin.

B. FLOW IN NATURAL CHANNELS

B–5. ***General.***—This portion of the appendix presents briefly the hydraulic theory and analyses of natural stream channel flow as related to the design of small dams. These analyses primarily involve the establishment of various hydraulic relationships from gathered field data that are applied to existing methods or procedures for deriving rating curves (stage-discharge relation) or making water surface profile computations. One of the more important uses of the rating curve is in connection with establishing tailwater conditions in the design of stilling basins.

The hydraulic analyses covered herein involve only the conditions of steady and nonuniform flow. A steady flow condition is said to prevail when the discharge is the same at all sections along the channel and remains constant with respect to time. The flow is said to be nonuniform when the grade of the water surface is different from that of the channel bottom, implying either accelerating or decelerating flow. Both conditions generally prevail in natural channels. The

flows are also considered to be contained only in the main channel portion of the stream. Reference is made to other publications [1, 8, 9] which contain the procedures for analyzing the hydraulics of streams where overbank flows occur.

Suggested procedures are given for the collection of an adequate set of field data to define each of the components in Manning's formula or Bernoulli's theorem which are used in the hydraulic computations. The engineer will have to select the most expedient and economical means of assembling the data and performing the computations for arriving at the answer. The accuracy of the results is dependent on gathering the field data representative of the prevailing hydraulic conditions and analyzing them through reasonable assumptions and interpretations. An accuracy of 0.5 foot is ordinarily a reasonable limit for the computations of water surfaces for low flows.

B-6. Collection of Data.—(a) *Streamflow Records.*—Records of streamflow taken from established United States Geological Survey gaging stations furnish valuable information for the required hydraulic analysis. A check with the nearest Geological Survey office or reference to the water supply papers published annually by this agency will show what streamflow data are available for each station (sec. 17). Detailed descriptions of gaging stations can be studied in Water Supply Paper 888 [10].

Damsites are seldom located near gaging stations. The rating curve of a station can be transposed to a cross section at or near the damsite if the hydraulic conditions of the reach between the damsite and the station are relatively uniform. A reasonable distance between the damsite and the gaging station for transposing the curve probably should not exceed 1,000 feet. Transpositions can be made either upstream or downstream from an established gaging station. Extreme changes in grade of the streambed, cross-sectional dimensions (particularly the width), and *n* values destroy the uniformity of the channel reach and consequently reduce the accuracy of the transposed rating curve.

(b) *Topographic Maps and Aerial Photographs.*—A topographic map and aerial photographs, if available (sec. 17), also provide useful information for rating curve development or water surface profile computations. For rough studies, the hydraulic properties of the stream at the damsite can be determined for a cross section as plotted from the topographic map. An example is included herein showing how this is done. The topographic map is also useful in studies involving water surface profile computations, where it can be used to locate a series of cross sections below the dam. Flow distances between such cross sections can be measured from the map.

Aerial photographs can be used to assist in selecting the location of the cross sections. Further, the *n* coefficients can be evaluated by observing the areal coverage of vegetation and the meandering pattern of the stream.

(c) *Field Surveys.*—The field work required to define the hydraulic dimensions of a single cross section is relatively simple and generally inexpensive. Essentially, the procedure involves setting up a level in direct line with the cross section and taking intermittent soundings across the section. Concurrently, levels can be run along the stream thalweg (lowest points in the stream-bed) or water surface (intermittent shots may be taken at water's edge along both banks) to define a slope for use in the Manning's formula; a distance of 200 to 300 feet downstream and upstream from the cross section will usually be sufficient. Elevations to the nearest tenth of a foot are satisfactory for cross-sectional data; however, they should be to the nearest hundredth of a foot for water surface profiles. This work should ordinarily be done in conjunction with topographic surveys of the damsite area.

If water surface profiles are to be computed for a more precise determination of a rating curve, a series of cross sections downstream from the dam will be needed. The costs for taking such field measurements are correspondingly higher than those for only one cross section; therefore, they may not be economically justified in certain instances.

High water marks, either observed or from recorded data, are used to define the water surface slope. Field observations of these marks should be made immediately following the occurrence of any flows of sufficient magnitude. An example of their use in a slope-area computation is shown in the next section.

(d) *Determination of n Values.*—The selection of *n* values for use in the Manning formula requires considerable judgment. Table B-6 gives

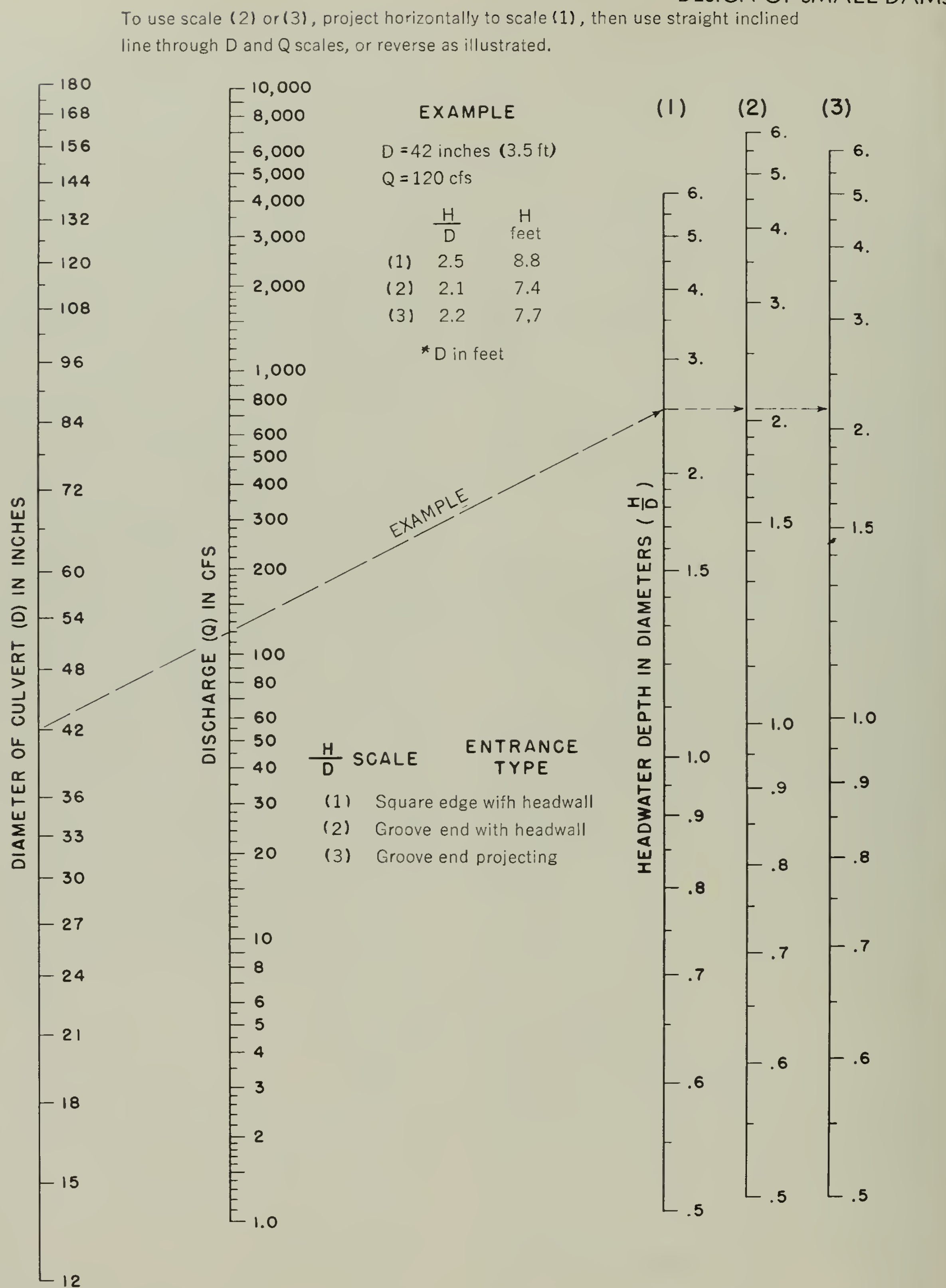


Figure B–8. Headwater depth for concrete pipe culverts with entrance control. (U.S. Bureau of Public Roads.)

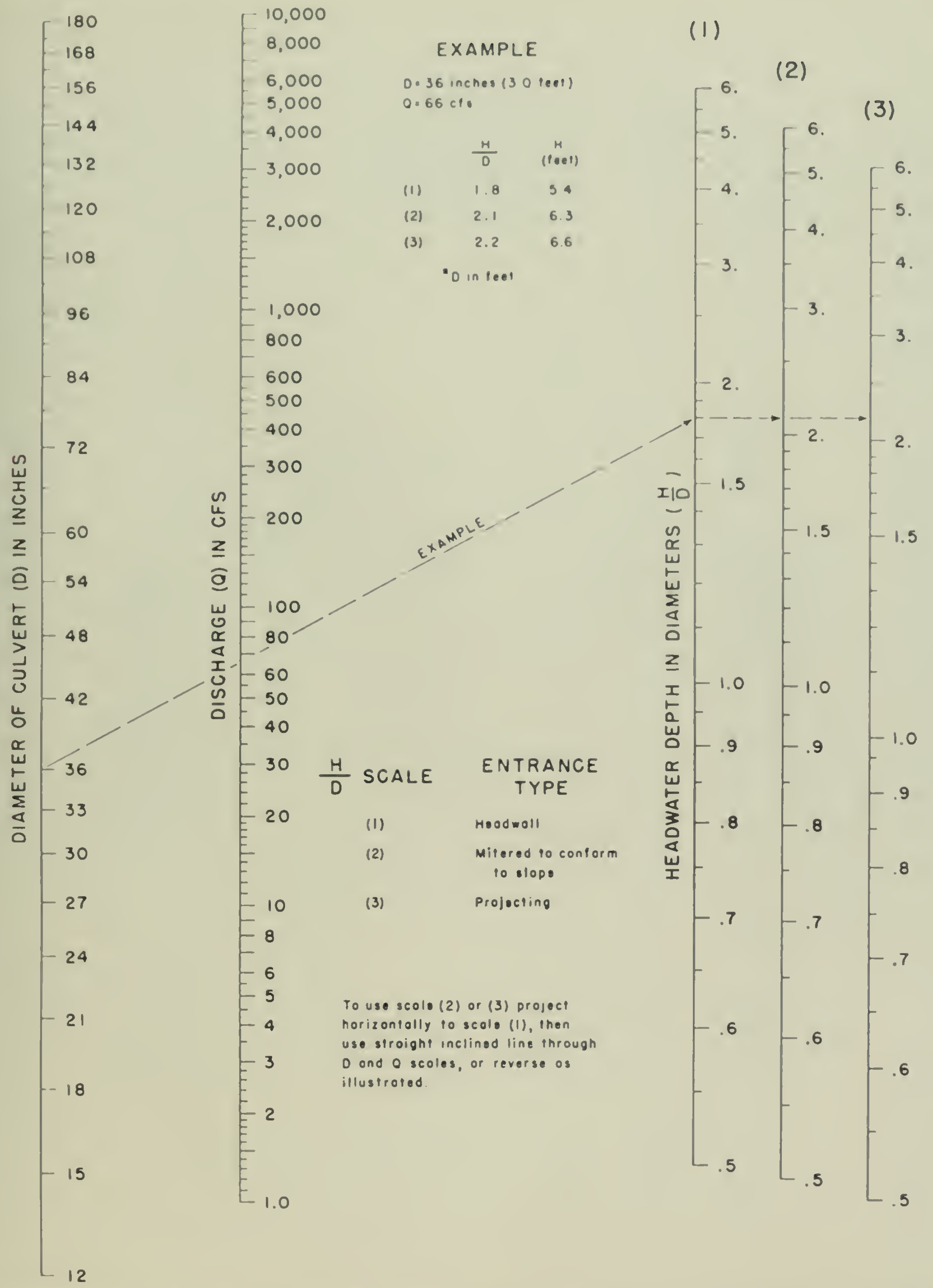
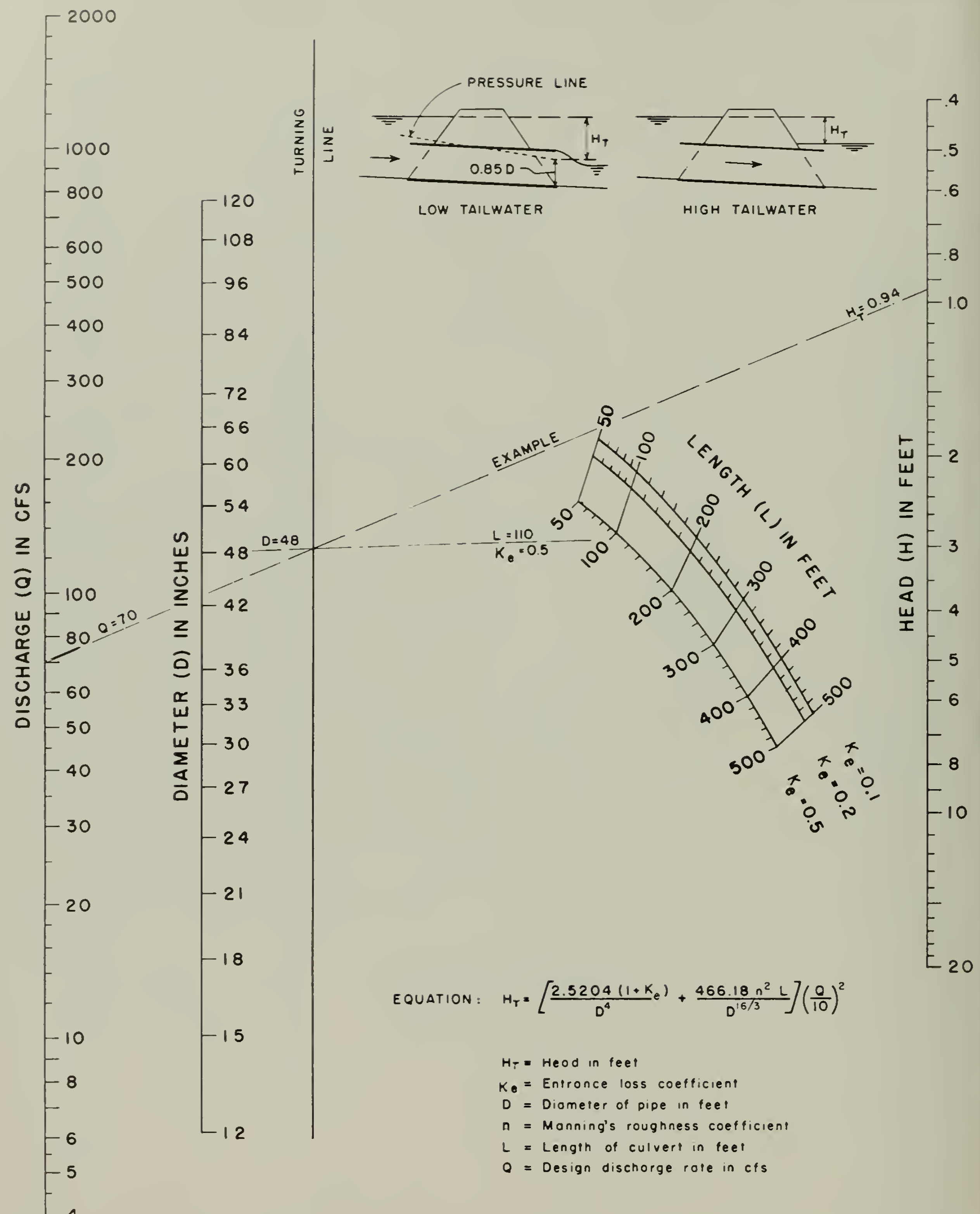


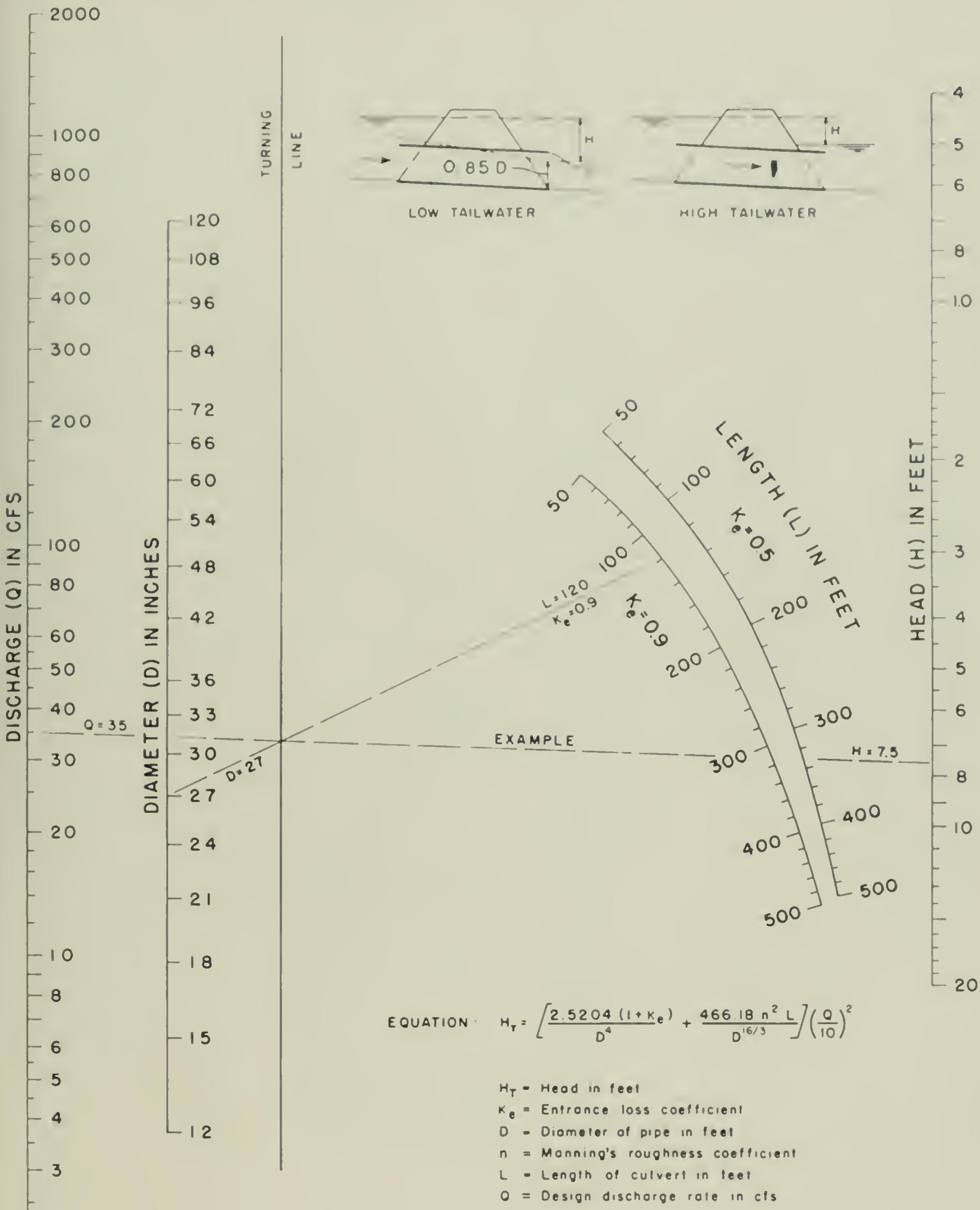
Figure B–9. Headwater depth for corrugated-metal pipe culverts with entrance control. (U.S. Bureau of Public Roads.)



EQUATION: $H_T = \left[\dfrac{2.5204\,(1+K_e)}{D^4} + \dfrac{466.18\,n^2\,L}{D^{16/3}}\right]\left(\dfrac{Q}{10}\right)^2$

H_T = Head in feet
K_e = Entrance loss coefficient
D = Diameter of pipe in feet
n = Manning's roughness coefficient
L = Length of culvert in feet
Q = Design discharge rate in cfs

Figure B-10. Head for concrete pipe culverts flowing full, n=0.012. (U.S. Bureau of Public Roads.)



EQUATION $H_T = \left[\frac{2.5204\,(1 + K_e)}{D^4} + \frac{466.18\,n^2 L}{D^{16/3}}\right]\left(\frac{Q}{10}\right)^2$

H_T = Head in feet
K_e = Entrance loss coefficient
D = Diameter of pipe in feet
n = Manning's roughness coefficient
L = Length of culvert in feet
Q = Design discharge rate in cfs

Figure B-11. Head for corrugated-metal pipe culverts flowing full, $n = 0.024$. (U.S. Bureau of Public Roads.)

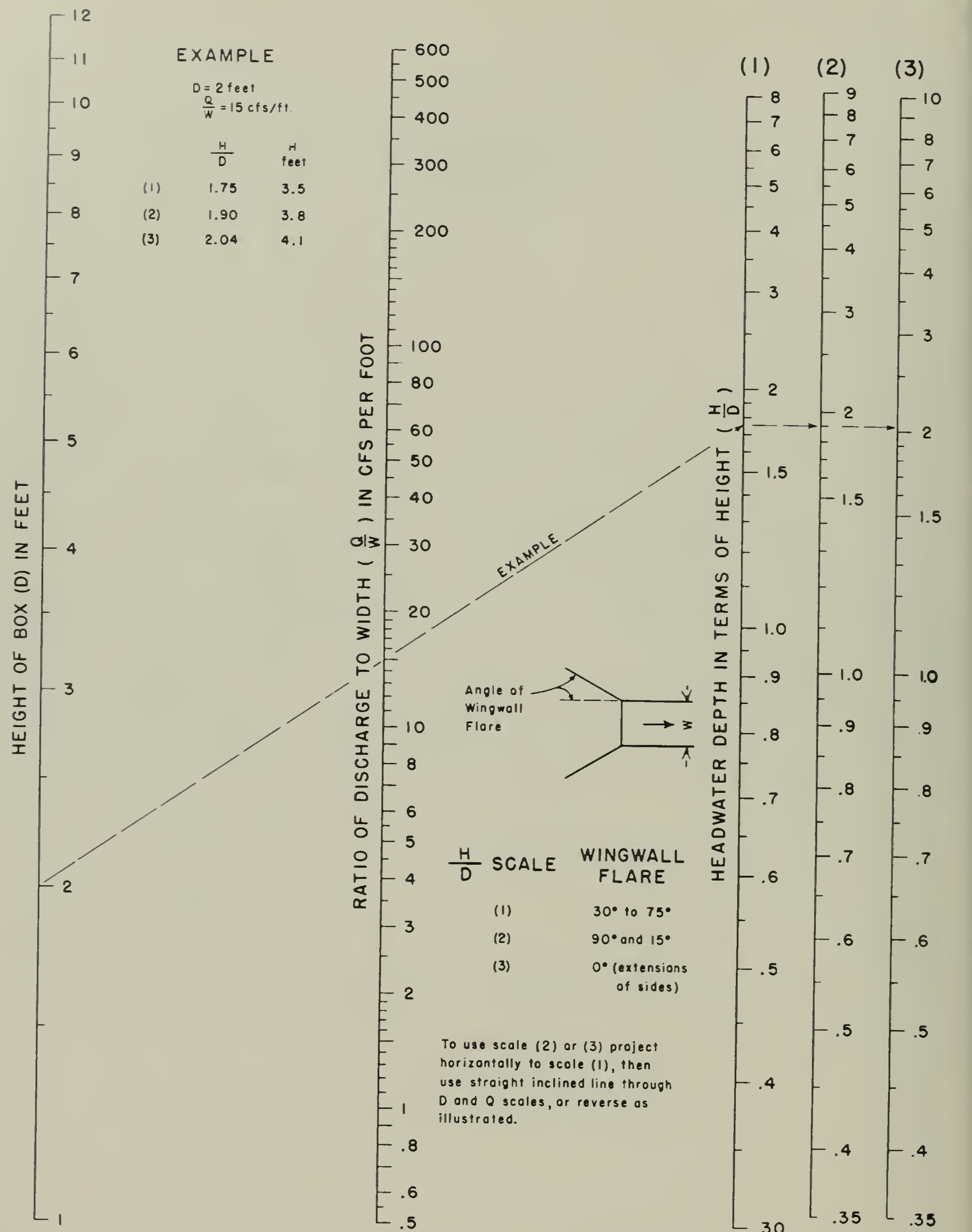


Figure B–12. Headwater depth for box culverts with entrance control. (U.S. Bureau of Public Roads.)

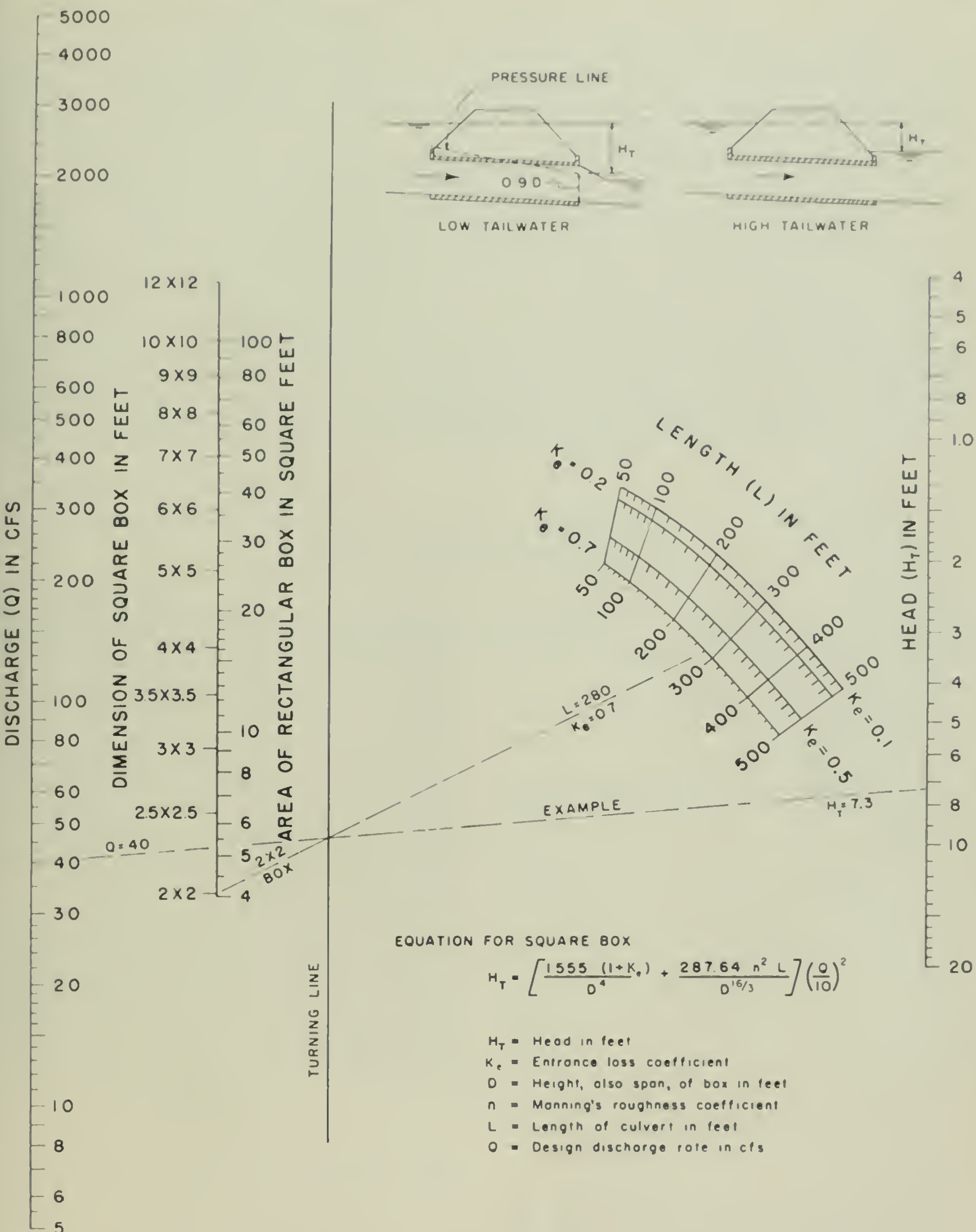


Figure B-13. Head for concrete box culverts flowing full, $n = 0.013$. (U.S. Bureau of Public Roads.)

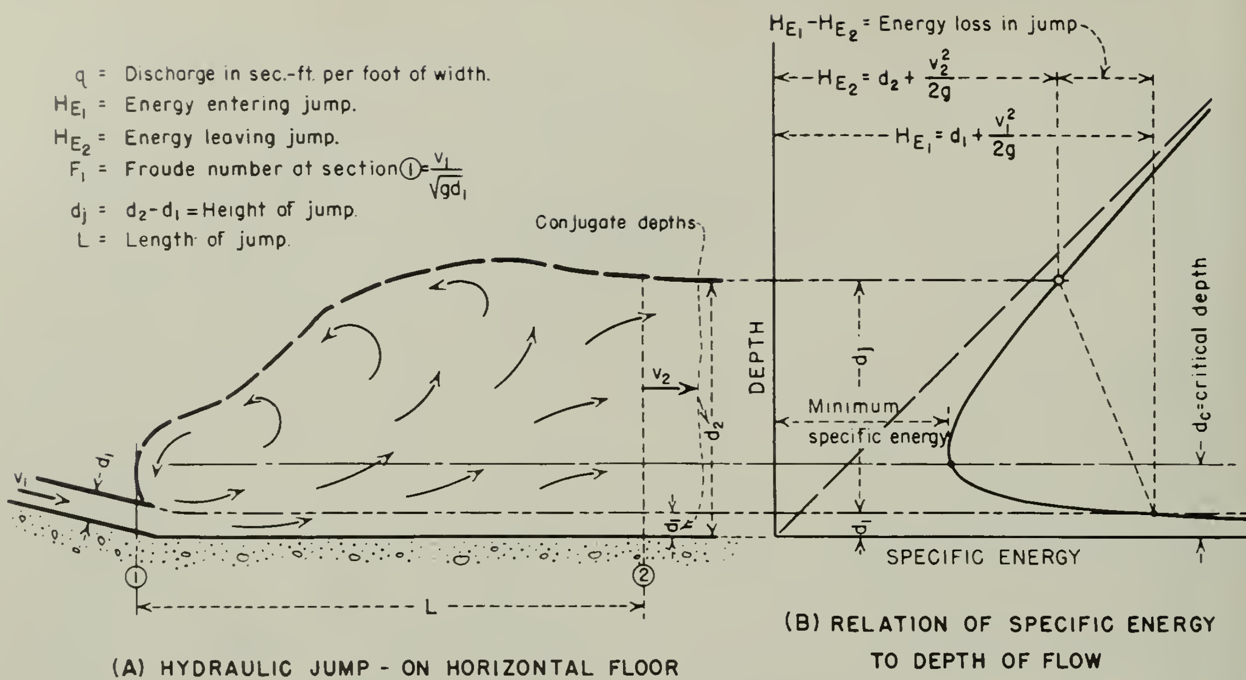


Figure B–14. **Hydraulic jump symbols and characteristics.**

values of n for average channels of various conditions. Table B–7 presents a procedure for computing a mean n value by systematically considering the factors which are involved.

The following publications will serve as guides in the proper selection of n values.

1. "Hydraulic and Excavation Tables," U.S. Department of the Interior, Bureau of Reclamation, eleventh edition, 1957, U.S. Government Printing Office, Washington, D.C.
2. "Handbook of Hydraulics," H. W. King, revised by E. F. Brater, fourth edition, 1954, McGraw-Hill Book Co., Inc., New York, N.Y.
3. "Hydrologic and Hydraulic Analyses, Computation of Backwater Curves in River Channels," Part CXIV, Chapter 9, Engineering Manual, Civil Works Construction, May 1952, Department of the Army, Corps of Engineers, Office of the Chief of Engineers.
4. "Flow of Water in Drainage Channels," C. E. Ramser, Technical Bulletin No. 129, November 1929, U.S. Department of Agriculture, Washington, D.C.
5. "Flow of Water in Irrigation and Similar Canals," F. C. Scobey, Bureau of Agricultural Engineering, U.S. Department of Agriculture, Washington, D.C., February 1939.
6. "Design Criteria for Interrelated Highway and Agricultural Drainage and Erosion Control," Tentative ASAE Recommendation, Agricultural Engineers Yearbook, 1958, American Society of Agricultural Engineers.

B–7. ***Slope-Area Method of Computing Streamflow.***—The slope-area method is utilized primarily to determine the discharge of a stream from specific field data. However, if the discharge is known, this method can be used to compute the value of n. Field procedures required to obtain needed data for the slope-area method include: selecting a representative reach of river channel; determining the channel cross sections at each end of the selected reach; measuring the water surface slopes from observed high water marks; and selecting a suitable roughness factor, n.

With these data, the discharge is determined by Manning's formula, equation (31), by a trial and error procedure. This procedure first involves combining such factors as the area, hydraulic radius, and n to compute the conveyance capacity, K_d, for each section, where the value of K_d is given by the equation:

$$K_d = \frac{1.486}{n} ar^{2/3} \tag{53}$$

From a comparison of equation (53) and equation (31), it may be seen that:

$$Q = K_d s^{1/2} \tag{54}$$

An approximate discharge is then computed for each section by multiplying the value of K_d by

TABLE B-6.—*Coefficient of roughness, average channels*

Value of n	*Channel condition*
0.016–0.017	Smoothest natural earth channels, free from growth, with straight alinement
0.020	Smooth natural earth channels, free from growth, little curvature.
0.0225	Average, well-constructed, moderate-sized earth channels in good condition
0.025	Small earth channels in good condition, or large earth channels with some growth on banks or scattered cobbles in bed.
0.030	Earth channels with considerable growth. Natural streams with good alinement, fairly constant section Large floodway channels, well maintained.
0.035	Earth channels considerably covered with small growth. Cleared but not continuously maintained floodways.
0.040–0.050	Mountain streams in clean loose cobbles. Rivers with variable section and some vegetation growing in banks. Earth channels with thick aquatic growths
0.060–0.075	Rivers with fairly straight alinement and cross section, badly obstructed by small trees, very little underbrush or aquatic growth.
0.100	Rivers with irregular alinement and cross section, moderately obstructed by small trees and underbrush. Rivers with fairly regular alinement and cross section, heavily obstructed by small trees and underbrush
0.125	Rivers with irregular alinement and cross section, covered with growth of virgin timber and occasional dense patches of bushes and small trees, some logs and dead fallen trees.
0.150–0.200	Rivers with very irregular alinement and cross section, many roots, trees, bushes, large logs, and other drift on bottom, trees continually falling into channel due to bank caving.

TABLE B-7.—*A method of computing mean n value for a channel*

(Used by U.S. Soil Conservation Service)

Steps

1. Assume basic n
2. Select modifying n for roughness or degree of irregularity
3. Select modifying n for variation in size and shape of cross section
4. Select modifying n for obstructions such as debris deposits, stumps, exposed roots, and fallen logs
5. Select modifying n for vegetation
6. Select modifying n for meandering
7. Add items 1 through 6

Aids in Selecting Various n Values

1. Recommended basic n values

Channels in earth	0.010	Channels in fine gravel	0.014
Channels in rock	0.015	Channels in coarse gravel	0.028

2. Recommended modifying n value for degree of irregularity

Smooth	0.000	Moderate	0.010
Minor	0.005	Severe	0.020

3. Recommended modifying n value for changes in size and shape of cross section

Gradual	0.000	Frequent	0.010 to 0.015
Occasional	0.005		

4. Recommended modifying n value for obstructions such as debris, roots, etc.

Negligible effect	0.000	Appreciable effect	0.030
Minor effect	0.010	Severe effect	0.060

5. Recommended modifying n values for vegetation

Low effect	0.005 to 0.010	High effect	0.025 to 0.050
Medium effect	0.010 to 0.025	Very high effect	0.050 to 0.100

6. Recommended modifying n value for channel meander

L_s = Straight length of reach $\quad$ L_m = Meander length of reach

L_m/L_s	n
1.0–1.2	0.000
1.2–1.5	0.15 times n_s
>1.5	0.30 times n_s

where n_s = items 1 + 2 + 3 + 4 + 5

the square root of the water surface slope obtained from measurements. Because of the difference in the cross-sectional areas of the upstream and downstream section, with consequent differences in the velocities and velocity heads, the average energy gradient obtained will not be parallel to the water surface. An alternative trial discharge must then be assumed, using an average value for K_d, until such values of gradient, velocity head, friction and other losses become consistent.

The foregoing procedure is illustrated in the following example using data taken by the U.S. Geological Survey on Touby Run at Mansfield, Ohio, for the flood of January 30, 1947:

Example 1

Problem: To determine the discharge.

Given: (a) Approximate bottom and observed high water surface profiles, and locations and cross-sectional plottings of section E (downstream) and section F (upstream), as shown in figure B–17.

(b) n for both sections equals 0.030.

(c) G=fall of water surface=0.22 foot

(d) L=length of reach=49 feet.

(e) s_{ws}=slope of water surface$=\frac{G}{L}=\frac{0.22}{49}$ =0.00449.

Solution: First trial computations are shown in table B–8.

TABLE B–8.—*First trial computations for example 1*

Section	a	p	r	$r^{2/3}$	n	(Eq. 53) K_d	s_{ws}	$s_{ws}^{1/2}$	$Q=K_d s_{ws}^{1/2}$
E (DS)	80.4	35.0	2.29	1.737	0.030	6900	0.00449	0.0671	463
F (US)	81.9	35.2	2.33	1.757	.030	7110	.00449	.0671	478

It is noted that the area of section E is smaller than that of section F, resulting in a gain in velocity head from F to E. When the velocity head at the downstream section is greater than that at the upstream section, the average slope of the energy gradient will be flatter than the water surface slope. The true discharge, therefore, must be less than that computed by the first trial. To proceed with the adjustment of the slope of the energy gradient to make it consistent with the velocity heads, the change of velocity heads $(h_{v_2}-h_{v_1})$ is added algebraically to the observed fall. This is illustrated by referring to figure B–18 where $G+h_{v_2}=h_f+h_{v_1}$. Solving for h_f,

$$h_f=G+(h_{v_2}-h_{v_1})$$

In this example, the channel is contracting and the turbulence is assumed suppressed; no head loss other than that due to friction is included. The full amount of the change in velocity head $(h_{v_2}-h_{v_1})$, therefore, is added algebraically. However, if the cross-sectional area of E had been greater than that of F, an expanding channel would exist. In this case expansion losses would be included, the amount depending upon the degree of expansion. If one-half of the change in velocity head is assumed lost,

$$h_L=G+0.5(h_{v_2}-h_{v_1})$$

To proceed with the example, alternative discharges are assumed by successive trials until

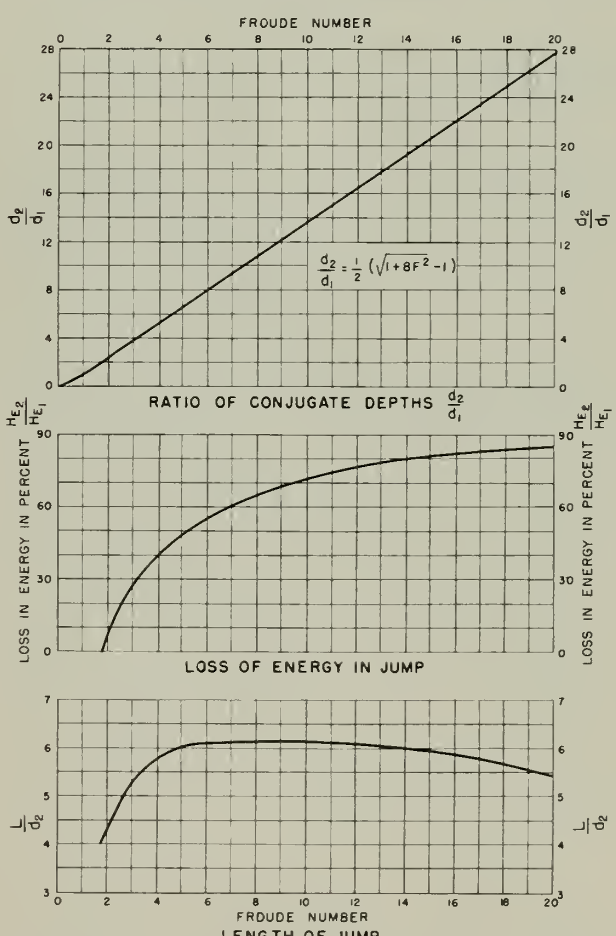


***Figure B–15.* Hydraulic jump properties in relation to Froude number.**

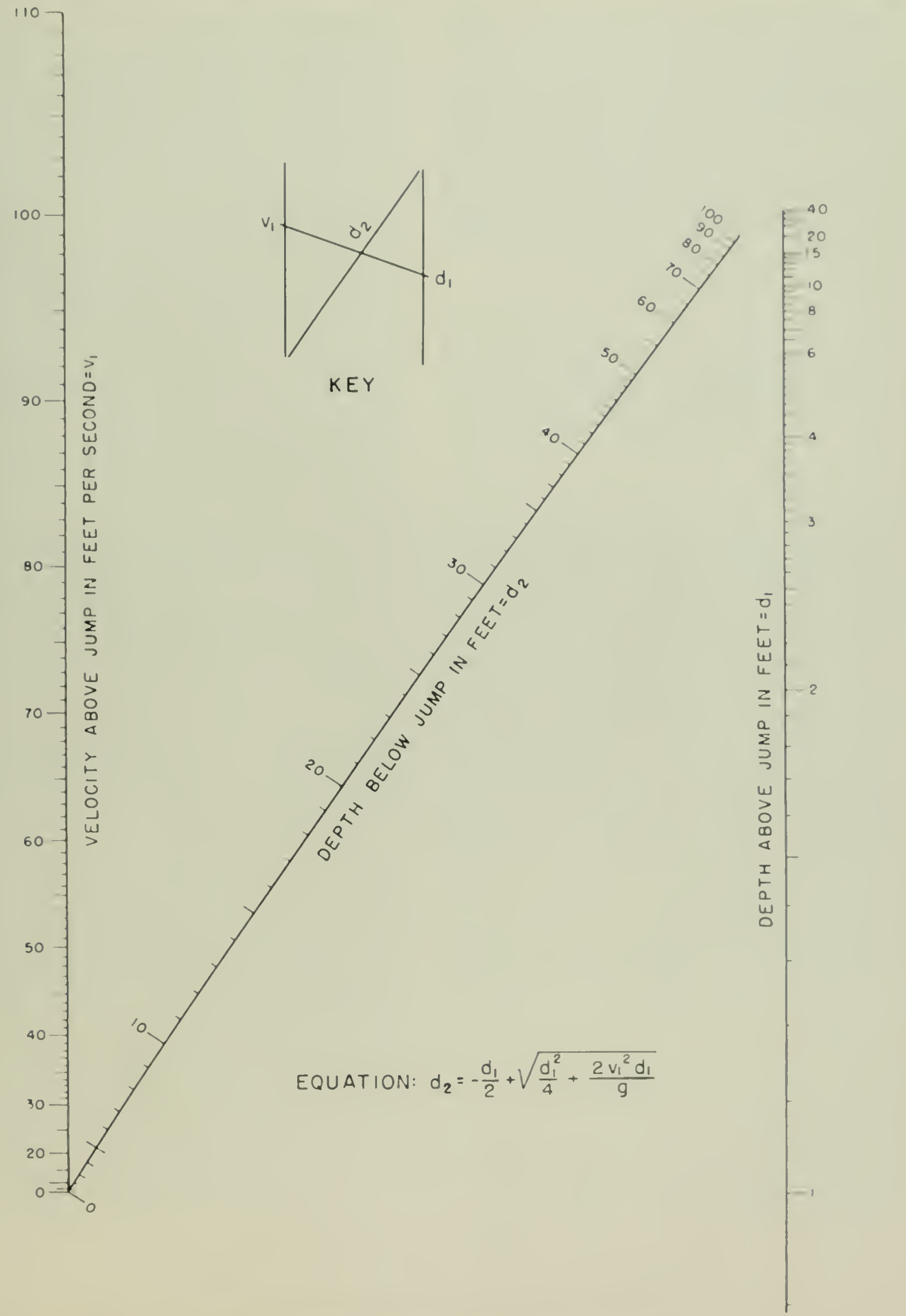


Figure B–16. Relation between variables in the hydraulic jump.

the assumed and computed discharges are equal. This process is illustrated in table B–9.

An analysis similar to that applied in the preceding slope-area method of computing discharge can also be used to determine an n value. This requires a known discharge such as may be available from the records of a gaging station within a reasonable distance from the site. To determine n, the Manning formula, equation (31), is expressed as follows:

$$n=\frac{1.486ar^{2/3}s_{ws}^{1/2}}{Q}$$

All the components on the right-hand side of the equation are determined by field measurements as in the preceding example. In this application the slope of the water surface can be reasonably substituted for that of the energy gradient, since the square roots of these slopes are approximately equal.

B–8. *Development of Rating Curves.*—(a) *Approximate Method.*—The field data required for development of a rating curve are similar to those used in a slope-area analysis. A determination must be made of the hydraulic properties for the section under consideration. This can be done by field measurements of the cross section by soundings; by a survey to determine either the water surface or channel bottom slopes; and by the assignment of an n value for the affected area.

When the economics of the project will not allow detailed field measurements to be taken, the development of the curve should be made from whatever pertinent data are available. A topographic map might be used for determining the cross section and possibly the channel bed profile. Other sources of information might provide data from which a selection of an n value can be made.

Figures B–19 and B–20 and table B–10 illustrate the preparation of a rating curve from a topographic map. The procedures shown are also applicable if the cross section of the stream is determined from field survey. First, the centerline of the dam was located on the topographic map available for the area, as shown in (A) of figure B–19. The cross section in (B) was then developed by scaling the distances between contours on the map. The low point in the streambed was determined by interpolation between contours. The mean bed slope of the stream was obtained from map-scaled measurements of distances between contours crossing the stream channel. A value of n of 0.030 was selected on the basis of various descriptions and field observations that a considerable growth of vegetation was present in a stream of relatively straight alinement. The computations were then performed as shown in table B–10.

The conveyance capacity method illustrated under the slope-area method has been utilized in the computations. From the hydraulic properties

TABLE B–10.—*Computations for tailwater rating curve*

ELK CREEK DAM

Data: $n=0.030$ $s_b=$ mean bed slope$=0.00395$

$$C=\frac{1.486}{n}=49.53 \quad s_b^{1/2}=0.0628$$

Elevation	a	p	r	$r^{2/3}$	C	K_d	$s_b^{1/2}$	Q
26.2	0	0	0	0				
30	95	50.6	1.88	1.523	49.53	7,170	0.0628	450
35	400	75.1	5.32	3.047	49.53	60,400	.0628	3,790
40	753	87.6	8.60	4.198	49.53	156,000	.0628	9,800
45	1160	99.2	11.70	5.154	49.53	296,000	.0628	18,600
50	1610	111.7	14.40	5.919	49.53	472,000	.0628	29,600

for the section and an n of 0.030, the conveyance capacity, K_d, is computed for various elevations. These are multiplied by the one-half power of the channel bottom mean slope to compute the discharge (equation (54)). The tailwater rating curve derived from the computations is shown in

TABLE B–9.—*Check computations for example 1*

[Using average K_d value from table B–8=7005]

Section	v	h_v	$(h_{v_2}-h_{v_1})$	$h_f=G+(h_{v_2}-h_{v_1})$	$s=\frac{h_f}{L}$	$s^{1/2}$	$K_{d_{avg.}}$	$Q=K_{d_{avg.}}s^{1/2}$
Assume $Q=460$ second-feet								
E	5.73	0.51						
F	5.62	.49	−0.02	0.20	0.00408	0.064	7005	448 Too small.
Assume $Q=450$ second-feet								
E	5.61	.49						
F	5.50	.47	−.02	.20	.00408	.064	7005	448 OK.

figure B–20.

(b) *Water Surface Profile Method.*—In studies where more exact tailwater curves are required, water surface profiles may be developed for various discharges. The computations in such studies are more involved and require a series of cross sections downstream from the dam site.

Numerous methods [1, 2, 8, 9] have been developed for computing water surface profiles; however, for a quick study Leach's method may be applied. This method is particularly adaptable to irregular channels flowing in reaches of relatively uniform channel bed slopes but not having excessive changes in cross-sectional characteristics. When these conditions exist, velocity head changes and turbulence losses can reasonably be neglected. Other methods [2, 8] are better suited to water surface profile computations accounting for velocity head changes and turbulence losses in Bernoulli's energy equation as applied to open channel flow.

In Leach's method a number of cross sections are selected at intervals below the dam site. Field data similar to those required for discharge computations by the slope-area method, as discussed in Section B-7, are required.

The following example shows the procedure required for establishing rating curve by Leach's method:

(1) Four sections designated A, B, C, D are selected at intervals along the stream, and the hydraulic properties of these sections are determined.

(2) A rating curve for the lowermost cross section, section A, is determined by the approximate method described in (a) of this section of the appendix.

(3) A conveyance curve is prepared for

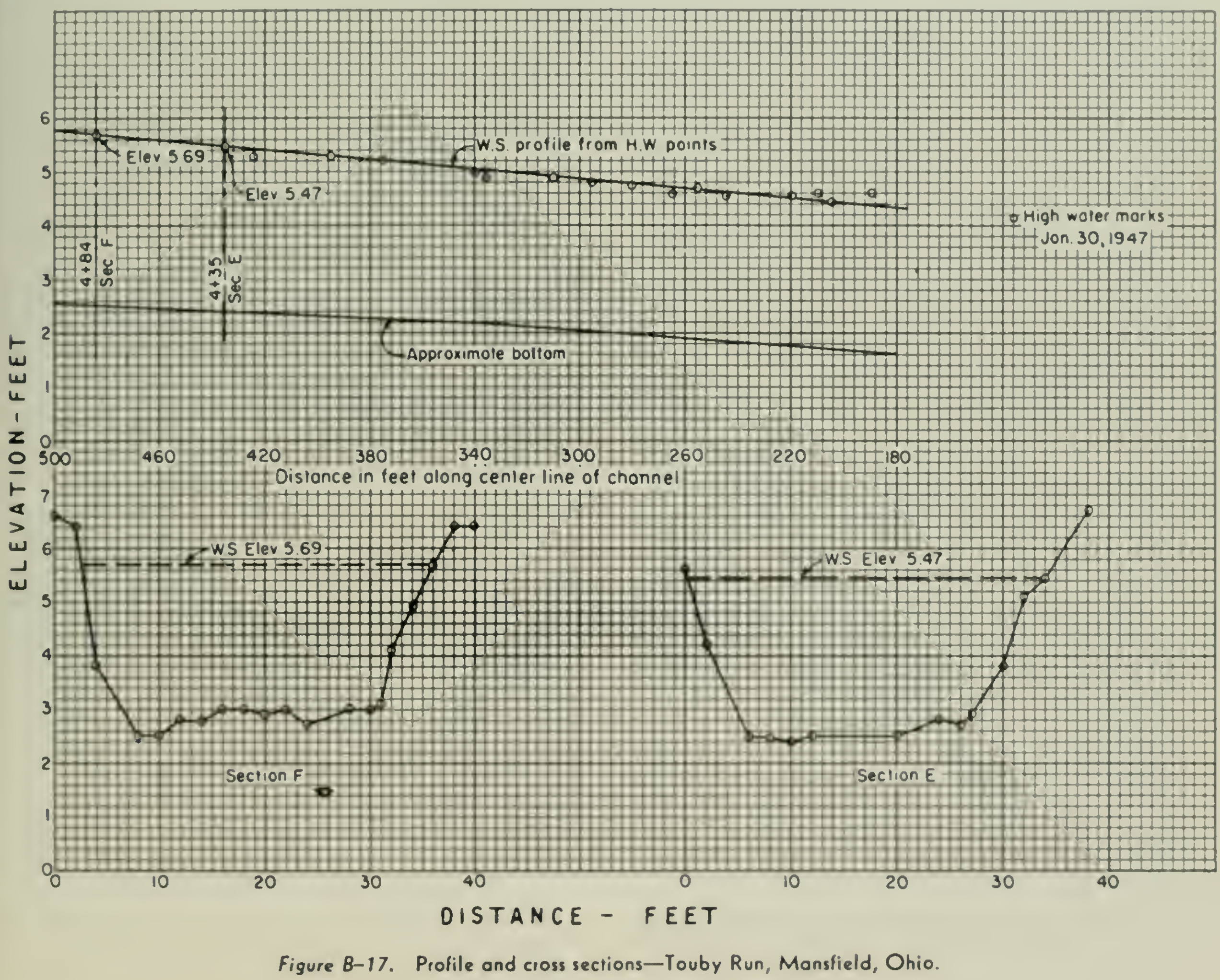


Figure B–17. Profile and cross sections—Touby Run, Mansfield, Ohio.

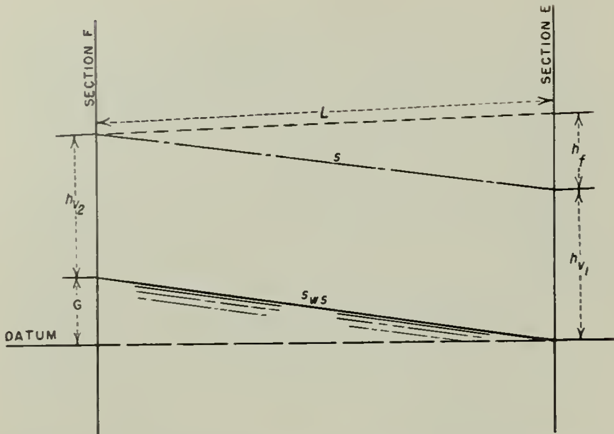


Figure B-18. **Bernoulli's theorem applied to the solution of example 1.**

each section, similar to the one shown in figure B-21 for section B.

(4) Water surface profiles are computed for at least four discharges to define the required tailwater curve. The computations shown in table B-11 are for $Q=500$ second-feet.

TABLE B-11.—*Water surface profile computation—Leach's method*

TICK TOCK CREEK

1 Station	2 Elevation	3 K_d	4 Avg. K_d	5 L	6 $s=\left(\frac{Q}{\text{avg. } K_d}\right)^2$	7 h_f	8 Elevation
For $Q=500$ second-feet							
Section A	1620.0	7,500					1620.0
			7,850	200	0.00406	0.81	
Section B	1620.8	8,200					1620.81
			8,350	300	.00359	1.08	
Section C	1621.9	8,500					1621.89
			8,100	350	.00382	1.34	
Section D	1623.2	7,700					1623.23

(5) Section A, the farthest downstream section, is the starting point of the profile computation. The water surface elevation 1620.0 in column 2 was taken from the previously determined rating curve for that section (not shown). The K_d value was read from the conveyance capacity curve at elevation 1620 for section A (not shown) and entered in column 3. The same elevation shown in column 2 is entered in column 8.

(6) A water surface is then assumed for the next section upstream, section B. In the example, it is assumed as 1620.8 and the corresponding K_d value is read from figure B-21 and listed in column 3.

(7) Column 4 shows the average K_d values of successive reaches between cross sections.

(8) The length of successive reaches between cross sections is entered in column 5.

(9) The slope of the energy gradient is computed from equation (54) as expressed in column 6, using the average K_d value of column 4.

(10) The head loss, h_f, in column 7, is computed by multiplying the values in columns 5 and 6.

(11) The elevation in column 8 is determined by adding the value of column 7 to the preceding amount of column 8. Thus in the example, for section B, $0.81+1620.0=1620.81$. The computations are complete when the

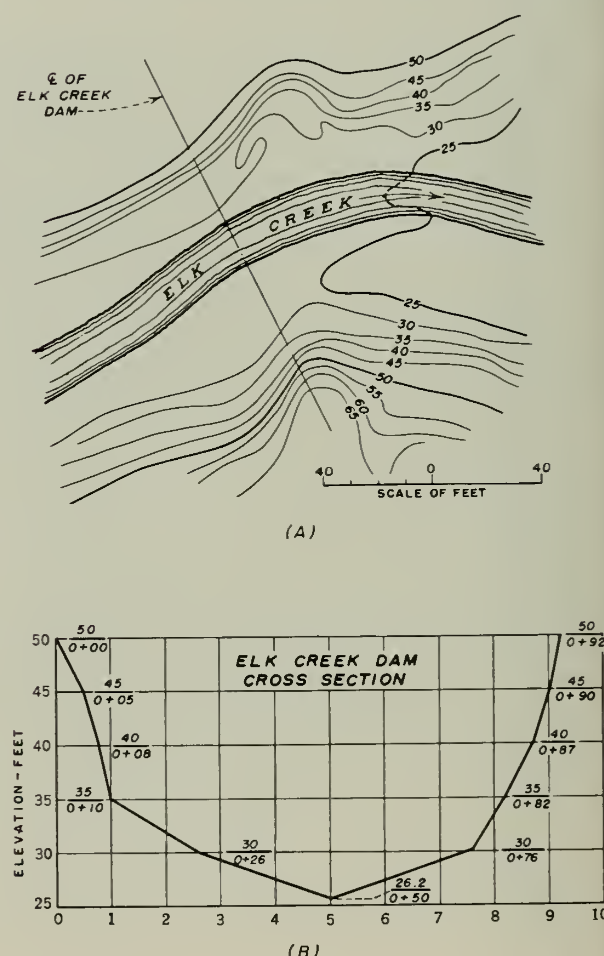


Figure B-19. **Plan and cross section of Elk Creek Dam site.**

elevations in columns 2 and 8 are equal. Generally, the balance is considered satisfactory when the difference in the values of columns 2 and 8 does not exceed 0.1 foot. If the balance is not obtained on the first try, other trial elevations are made until a satisfactory answer results. The computations then proceed to the next upstream station, section C, and the same procedure is repeated beginning with a trial elevation at that section.

(12) The computations are repeated for at least three other discharges to develop a number of water surface profiles.

(13) The rating curve for section D is plotted from the various water surface profiles.

This method of determining a rating curve is more reliable than the approximate method previously described, because if section A does not typify the average of the stream regime the variations are recognized as the profiles are continued upstream. This may be demonstrated by changing the rating curve used for section A in the above example and repeating the computations. It will be found that a considerable change in the rating curve for section A will make less difference in the rating curve for section D.

B-9. ***Critical Flow.*** The hydraulic analysis of flow in open channels becomes more complex when critical conditions can occur at some point along the river reach under consideration. The conditions of critical flow can be commonly observed at a "control" section in the channel. Such controls occur at locations where there is a material change in the cross section causing a constriction of the flow. These constrictions may be natural, or artificial such as bridges. Another cause may be significant changes in the bottom grade. The flow observed below critical sections is usually in a mild or possibly high turbulent state depending on the degree of control. Vortices, eddies, cross currents, and large standing waves are some of the characteristics indicating such flow conditions. A field reconnaissance of the hydraulic reach under investigation should include the location of any critical sections.

The probable existence of critical flow at a section is evident in water surface profile computations when a balance of Bernoulli's energy equation cannot be attained. A preliminary check for critical flow can be made by comparing the trial velocity with the critical velocity, as given in equation (14). When the velocity exceeds v_{cr}, it can be assumed that a control exists. Other methods may be used for checking critical flow, but they are not discussed, since all are based on different ways of analyzing equation (7).

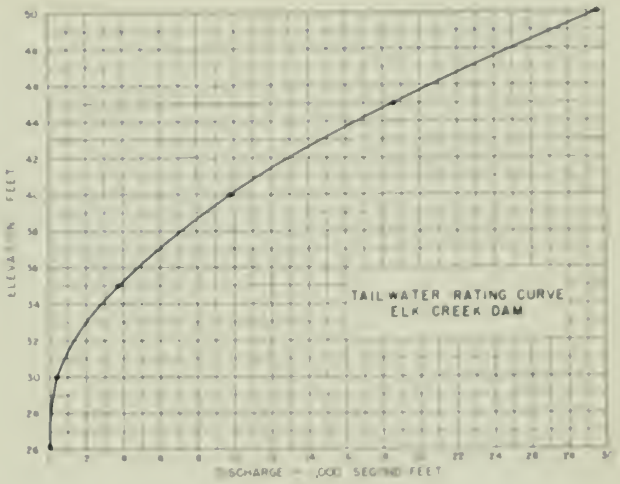


Figure B-20. Tailwater rating curve for Elk Creek Dam.

When the depth of flow is greater than critical depth throughout the reach of channel under consideration, and a control point either upstream or downstream from the reach is not evident or suspected, the normal procedure is carried out—a rating curve is developed for the lowermost section and computations for water surface profiles are made proceeding upstream.

If a control point is downstream from the reach, the computations should begin at this point and proceed upstream. When the control point is within the reach, the critical depth elevation is determined and the hydraulic properties of the cross section where this elevation occurs are used to continue the computations upstream.

A control point upstream from the reach can become a special problem depending on the degree of influence it has on the downstream flow conditions. If its influence is effective beyond the next downstream section such that a hydraulic jump occurs somewhere in the reach, the analysis necessary to define the water surface profiles is beyond the scope of this appendix. Generally, in

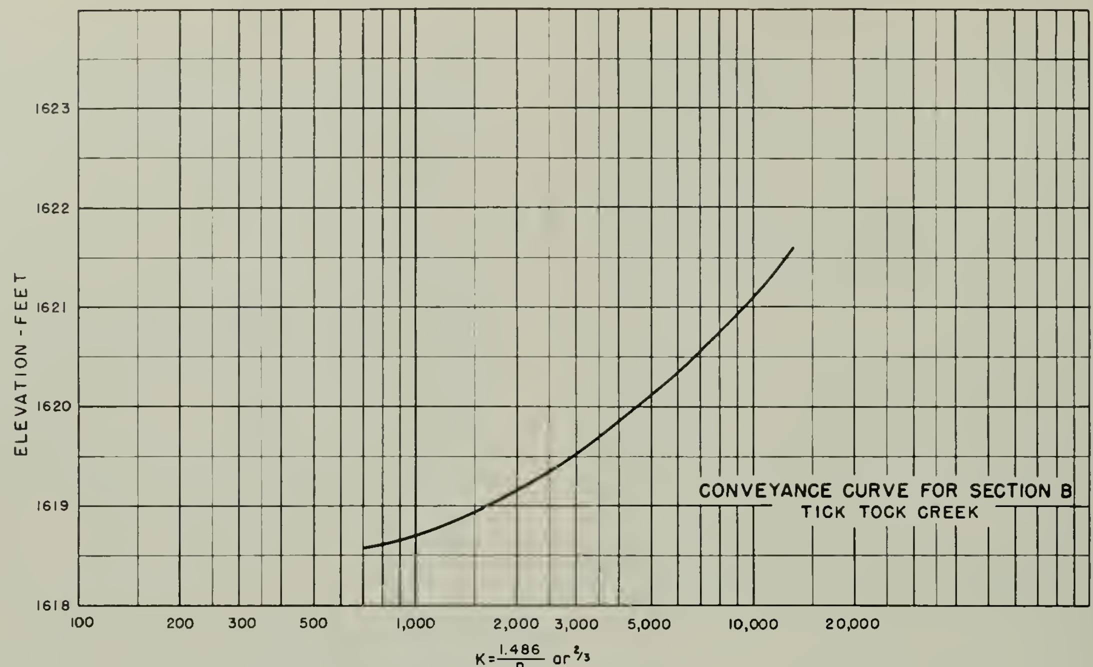


Figure B-21. **Conveyance curve for section B—Tick Tock Creek.**

most river stretches the flow conditions revert to the tranquil state as observed at the next downstream section, indicating the flow depth has become greater than the critical depth. Standing waves, vortices, and turbulent eddies are characteristic of the flow below this control point. With these conditions prevailing, the elevations for critical flow at this point may be used as the upstream limit of the computations for the water surface profiles.

The above discussion involves the analysis of critical flow as applied to water surface profile computations. It may be required to develop a critical rating curve for a control section which is located at or near the dam site. In this event, the critical velocities are computed by equation (14) and multiplied by the area to determine the discharge. This is done for several elevations and a curve of critical flow stage versus discharge plotted.

C. BIBLIOGRAPHY

B-10. *Bibliography.*

[1] King, H. W., revised by E. F. Brater, "Handbook of Hydraulics," fourth edition, McGraw-Hill Book Co., Inc., New York, N.Y., 1954.

[2] Woodward, S. B., and Posey, C. J., "Steady Flow in Open Channels," John Wiley & Sons, Inc., fourth printing, September 1949.

[3] Bakhmeteff, B. A., "Hydraulics of Open Channels," McGraw-Hill Book Co., Inc., New York, N.Y., 1932.

[4] Binder, R. C., "Fluid Mechanics," Prentice-Hall, Inc., Englewood Cliffs, N.J., third edition, 1955.

[5] Rouse, Hunter, "Engineering Hydraulics," John Wiley & Sons, Inc., New York, N.Y., 1950.

[6] Bradley, J. N., and Thompson, L. R., "Friction Factors for Large Conduits Flowing Full," Engineering Monograph No. 7, U.S. Department of the Interior, Bureau of Reclamation, March 1951.

[7] Bradley, J. N., and Peterka, A. J., "The Hydraulic Design of Stilling Basins," ASCE Proceedings, vol. 83, October 1957, Journal of Hydraulics Division, No. HY5, Papers No. 1401 to 1404, inclusive.

[8] "Guide for Computing Water Surface Profiles," U.S. Department of the Interior, Bureau of Reclamation, November 1957.

[9] "Hydrologic and Hydraulic Analyses—Computation of Backwater Curves in River Channels," Engineering Manual, Department of the Army, Corps of Engineers, pt. CXIV, ch. 9, May 1952.

[10] "Stream Gaging Procedure," Water Supply Paper 888, U.S. Department of the Interior, Geological Survey, 1945.

[11] Ramser, C. E., "Flow of Water in Drainage Channels," Technical Bulletin No. 129, U.S. Department of Agriculture, 1929.

Structural Design Data

C. J. HOFFMAN AND P. K. BOCK [1]

C–1. ***Introduction.***—This appendix presents structural design data peculiar to hydraulic structures for the design of concrete appurtenances to small dams. It is presumed that the user of this text is familiar with reinforced concrete design of a general nature or that he will consult other texts for information on this subject. A major portion of this appendix is concerned with the design of reinforced concrete conduits for use as spillways or outlet works under or through small earthfill dams.

C–2. ***Earth Pressures on Retaining Walls.***—Figure C–1 presents a method for obtaining active earth loads on retaining walls when the properties of the fill material behind the wall are known. The curves are based on Coulomb's theory of active earth pressure against retaining walls [1].[2] In applying Coulomb's theory, the angle of friction between the earth and the back of the wall has been assumed equal to zero. Detailed discussions of methods available for determining earth pressures and designing retaining walls can be found in many texts [1, 2, 3, 4]. Complete designs for retaining walls under 10 feet in height are given in a Portland Cement Association bulletin [5].

C–3. ***Allowable Bearing Values for Structure Footings.***—Table C–1 gives suggested allowable bearing values for footings of structures appurtenant to small dams. These values are based on an evaluation of several sources of published data [1, 6, 7], in the light of the problems peculiar to hydraulic structures. The allowable bearing values listed for foundations of soils are smaller than are generally given in building codes and, with the exception of the gravels, vary according to the relative density and relative consistency of cohesionless and cohesive soils, respectively, rather than with the soil classification group.

C–4. ***Precast Concrete Pipe Conduits.***—(a) *General.*—When precast concrete pipes are used for outlet works conduits (or for spillways) under or through earthfill dams, they should be bedded in a concrete base as shown on figure C–2. The primary purpose of the concrete base is to prevent percolation along the underside of the pipe where tightly compacted earth bedding is difficult to obtain. For this reason the concrete base should be placed concurrently with or after the pipe is in position.

Cutoff collars should be provided to prevent percolation along the pipe. These collars should be identical to those used with cast-in-place concrete conduits, as discussed in section 235(b).

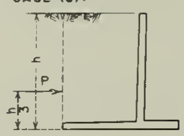
Precast concrete pipe joints should have rubber gaskets and the joints should not be filled with grout, so that greater flexibility will be provided should settlements due to superimposed embankment loads occur. For similar reasons joints should be provided in the concrete base. The longitudinal reinforcement should not extend through the joints in the base, and the joint locations should coincide with the pipe joints.

(b) *Design of Precast Concrete Pipe.*—When precast concrete pipe is used for outlet works, it should be designed for internal pressure, superimposed embankment loads, and a combination of the two as conditions require. The embankment loads should be computed in accordance with Marston's theory, as described in section 235(d). The precast concrete pipe should be assumed as a rigid conduit. In determining embankment loads for design purposes, the projection condition is

[1] Engineers, Spillways and Outlet Works Section, Bureau of Reclamation

[2] Numbers in brackets refer to items in the bibliography, section C–6.

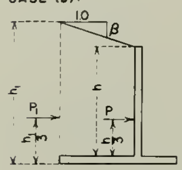
CASE (a):



$P = K_a \frac{wh^2}{2}$

where K_a is obtained from chart below, for β is 0 and w is the weight of backfill in lbs per cubic foot.

CASE (b):

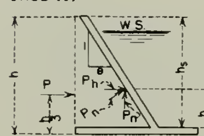


$P = K_a \frac{wh^2}{2}$

$P_1 = K_a \frac{wh_1^2}{2}$

where K_a is obtained from chart below, for $\beta > 0$

CASE (c):



P same as for case (a)

$P_n = K_a \frac{wh^2}{2}$

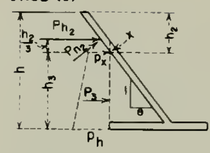
where K_a is obtained from chart below, for appropriate value of θ

$P_h = \frac{P_n}{\sqrt{1+\theta^2}}$

$P_v = \frac{\theta P_n}{\sqrt{1+\theta^2}}$

If stem is subjected to hydrostatic load on the channel side, assume fill has shrunk away from wall and check stem design for hydrostatic load only, using allowable steel stress of $1.4\ f_s$

CASE (d):



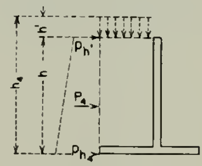
P_{h_2} and P_{n_2} computed above point x as in case (c)

$P_3 = \left(\frac{p_x + p_h}{2}\right) h_3$ where

$p_x = K_a w h_2$ and $p_h = K_a wh.$

P_3 is applied at $\frac{h_3}{3}\left(\frac{2p_x + p_h}{p_x + p_h}\right)$ above base

CASE (e):



For uniform surcharge, convert to equivalent height of fill h'.

Then $P_4 = \left(\frac{p_{h_4} + p_{h'}}{2}\right) h$ where

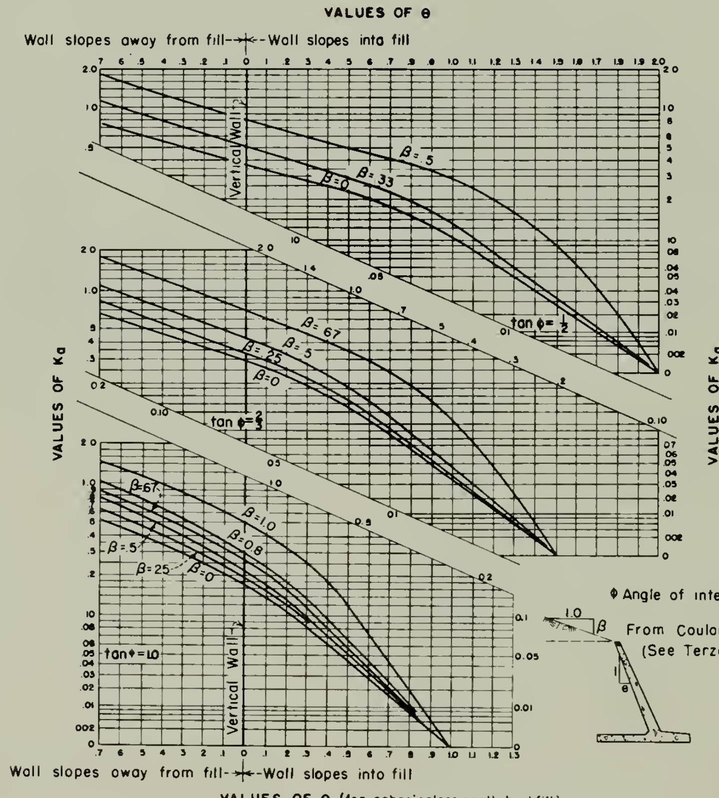
$p_{h_4} = K_a w h_4$ and $p_{h'} = K_a wh'$

P_4 is applied at $\frac{h}{3}\left(\frac{2p_{h'} + p_{h_4}}{p_{h'} + p_{h_4}}\right)$ above base.

NOTE: For fully saturated pervious backfill or backfill retaining water:

Vertical unit weight = unit weight of dry fill plus 62.5 lbs per cu ft x percent of voids in the fill

Horizontal unit pressure p = full hydrostatic pressure plus 0.4 of dry equivalent fluid pressure.



Φ Angle of internal friction

From Coulomb's equations (See Terzaghi:-"Theoretical Soil Mechanics" p 80)

Pressure coefficients for earthfill loads on walls

Figure C–1. **Earth pressures on retaining walls.**

TABLE C-1.—*Suggested allowable bearing values for footings of structures appurtenant to small dams*

Material	Condition, relative density, or relative consistency	Average standard penetration values in fine-grained soils		Allowable bearing pressure, tons per square foot
		Effective overburden pressure, pounds per square inch	Number of blows per foot	
Massive igneous metamorphic or sedimentary hard rock	Sound (minor cracks allowed)			100
Hard laminated rock such as slate	Sound (minor cracks allowed)			35
Residual deposits of bedrock of any kind except shale (Unsound shale is treated as clay.)	Shattered or broken			10
Gravel (GW, GP, GM, GC)				4
Cohesionless sands (SW, SP)	Loose	0 20 40	4 12 17	(2)
	Medium	0 20 40	4 to 8 12 to 24 17 to 40	1
	Dense	0 20 40	8 24 40	2
Saturated [1] cohesive sands, silts, and clays (SM, SC, ML, CL, MH, CH).	Soft Medium Stiff Hard		4 4 to 10 11 to 20 20	0.25 0.5 1.0 1.5

[1] Values are for foundations that are almost or completely saturated during the construction period. Bearing values can be increased by one-third if the foundation is relatively dry, provided that the criteria given in fig. 113 for "no treatment required" are met.

[2] Requires compaction.

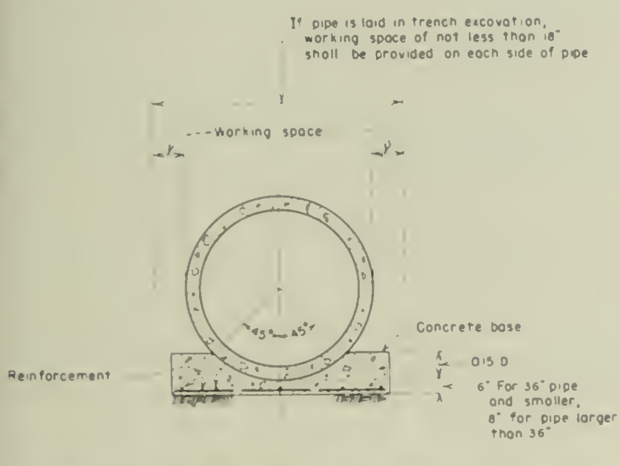


Figure C-2. **Precast concrete pipe on concrete base for conduit under or through earthfill dams.**

most likely to govern. The various conditions of densities of material and settlements in foundation and embankment must be given due consideration. It is well to remember that even where natural ground and embankment have equal densities, the settlements in constructed embankment will be greater than in the natural foundation.

Precast concrete pipe and reinforcement should be designed in accordance with the formulas and unit stresses given below. The formulas for the moments, shears, and thrust are based on bulb-like distribution of earth pressures [8]. Figure C-3 shows the location of the critical design sections and the notations. Dimensions r, r_1, t, and H in the formulas are in feet; W = total earth load on the pipe in pounds per linear foot, M = moment in foot-pounds, S = shear in pounds, and T = thrust in pounds.

Section 1: $M = 115r^2t - 24rr_1^2 - 0.126rW$

$T = +195rt - 53r_1^2 + 0.324W$

Section 2: $M = +83r^2t + 17rr_1^2 + 0.089rW$

$T = +280rt - 12r_1^2 + 0.539W$

Section 3: $S = -244rt - 51r_1^2 - 0.273W$

T due to hydrostatic pressure = $-62.4r_1H$

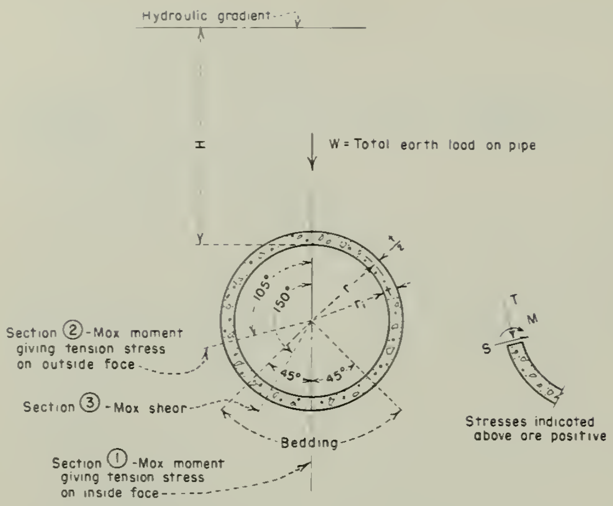


Figure C–3. **Location of critical sections in design of precast concrete pressure pipe.**

The pipe shell thickness and the reinforcement should be proportioned on the basis of the following design stresses:

Concrete	*Design stress, pounds per square inch*		
Ultimate strength of concrete, f_c'	3,500	4,500	6,000
Flexure, allowable extreme fiber stress in compression, f_c	1,575	2,025	2,700
Shear v	105	135	135
Bond, plain bars u	210	250	250
Bond, deformed bars u	350	350	350

Reinforcing steel: Reinforcement bars should conform to Federal Specification QQ–B–71a, type A or B. Allowable tensile stress f_s=20,000 pounds per square inch for combined moment and thrust when part of the section is in compression. When entire section is in tension, or when considering bursting stresses due to hydrostatic pressure alone, f_s should not exceed 16,000 pounds per square inch for hydrostatic heads up to 50 feet.

Concrete pipe should be manufactured in accordance with ASTM Specifications (Designation C361–57T). The pipe shell thickness selected should be in accordance with these specifications since most manufacturers have pipe forms for the thicknesses specified therein. Should design requirements require shell thicknesses greater than available for precast concrete pipe, it would be desirable to use cast-in-place type of construction.

Table C–2 gives minimum hoop reinforcement and nominal wall thickness for reinforced concrete pressure pipe from 12 inches to 96 inches in diameter and for overfills of up to 20 feet above the top of the pipe. In each case the internal pressure is taken as equal to 25 feet of head measured to the centerline of the pipe. Although this table was developed for the design of precast concrete pressure pipelines, it may be used for the design of precast concrete pipe conduits under earthfill dams.

TABLE C–2.—*Reinforcement and wall thicknesses for reinforced concrete pressure pipe for 12-inch through 96-inch diameter*

	Minimum hoop reinforcement in square inches per linear foot of barrel											
Internal diameter of pipe in inches	12		15		18				21			
Type of reinforcement	Circular		Circular		Circular		Elliptical		Circular		Elliptical	
Nominal wall thickness, inches	2	3	2	3	2¼	3	2¼	3	2⅜	3	2⅜	3
Layers of reinforcement	Single	Single	Single	Single	Single	Single	Single	Single	Single	Single	Single	Single
CLASS												
A–25	0.10	0.10	0.12	0.12	0.15	0.15	0.24	0.24	0.18	0.18	0.27	0.27
B–25	.10	.10	.14	.12	.18	.15	.24	.24	.23	.19	.27	.27
C–25	.13	.10	.19	.14	.24	.19	.24	.24	.32	.26	.27	.27
D–25	.16	.11	.25	.17	.32	.24	.25	.24	.42	.32	.31	.27

TABLE C-2.—*Reinforcement and wall thicknesses for reinforced concrete pressure pipe for 12-inch through 96-inch diameter*—Continued

	Minimum hoop reinforcement in square inches per linear foot of barrel—Continued													
Internal diameter of pipe in inches	24				27					30				
Type of reinforcement	Circular		Elliptical		Circular			Elliptical		Circular			Elliptical	
Nominal wall thickness, inches	2½	3	2½	3	2⅝	3¼		2⅝	3¼	2¾	3½		2¾	3½
Layers of reinforcement	Single	Single	Single	Single	Single	Inner	Outer	Single	Single	Single	Inner	Outer	Single	Single
CLASS														
A-25	0.20	0.20	0.32	0.32	0.24	0.17	0.12	0.35	0.35	0.28	0.18	0.13	0.39	0.39
B-25	29	26	32	32	35	19	11	35	35	41	21	12	39	39
C-25	40	33	32	32	50	25	13	35	35	60	28	15	39	39
D-25	54	43	37	32	68	32	16	44	35		35	18	51	39

	Minimum hoop reinforcement in square inches per linear foot of barrel									
Internal diameter of pipe in inches	33					36				
Type of reinforcement	Circular			Elliptical		Circular			Elliptical	
Nominal wall thickness, inches	2⅞	3¾		2⅞	3¾	3⅛	4		3⅛	4
Layers of reinforcement	Single	Inner	Outer	Single	Single	Single	Inner	Outer	Single	Single
CLASS										
A-25	0.36	0.25	0.19	0.43	0.43	0.44	0.31	0.25	0.47	0.47
B-25	48	27	17	43	43	53	34	22	47	47
C-25	71	.31	17	45	43	79	36	20	50	47
D-25		39	20	59	43		45	23	66	47

	Minimum hoop reinforcement in square inches per linear foot of barrel											
Internal diameter of pipe in inches	39						42					
Type of reinforcement	Circular				Elliptical		Circular				Elliptical	
Nominal wall thickness, inches	3½		4¼		3½	4¼	3¾		4½		3¾	4½
Layers of reinforcement	Inner	Outer	Inner	Outer	Single	Single	Inner	Outer	Inner	Outer	Single	Single
CLASS												
A-25	0.27	0.20	0.33	0.28	0.51	0.51	0.28	0.22	0.36	0.30	0.55	0.55
B-25	37	22	37	24	51	51	39	23	39	27	55	55
C-25	50	28	39	22	51	51	53	30	43	23	55	55
D-25	65	36	49	25	65	51	69	37	53	28	69	55

	Minimum hoop reinforcement in square inches per linear foot of barrel											
Internal diameter of pipe in inches	45						48					
Type of reinforcement	Circular				Elliptical		Circular				Elliptical	
Nominal wall thickness, inches	3⅞		4¾		3⅞	4¾	4⅛		5		4⅛	5
Layers of reinforcement	Inner	Outer	Inner	Outer	Single	Single	Inner	Outer	Inner	Outer	Single	Single
CLASS												
A-25	0.32	0.24	0.39	0.32	0.59	0.59	0.35	0.27	0.42	0.34	0.62	0.62
B-25	42	25	42	29	59	59	47	29	46	30	62	62
C-25	58	34	46	25	59	59	67	38	52	28	68	62
D-25	76	42	57	30	76	59	88	49	66	35	80	65

TABLE C-2.—*Reinforcement and wall thicknesses for reinforced concrete pressure pipe for 12-inch through 96-inch diameter*—Continued

	Minimum hoop reinforcement in square inches per linear foot of barrel											
Internal diameter of pipe in inches	51						54					
Type of reinforcement	Circular				Elliptical		Circular				Elliptical	
Nominal wall thickness, inches	4¼		5¼		4¼	5¼	4½		5½		4½	5½
Layers of reinforcement	Inner	Outer	Inner	Outer	Single	Single	Inner	Outer	Inner	Outer	Single	Single
CLASS												
A-25	0.39	0.29	0.45	0.37	0.67	0.67	0.42	0.32	.48	0.40	0.70	0.70
B-25	.51	.31	.49	.33	.67	.67	.53	.32	.52	.36	.70	.70
C-25	.74	.42	.55	.30	.74	.67	.76	.43	.59	.32	.76	.70
D-25	.97	.53	.70	.37	.97	.70	1.00	.55	.74	.39	1.00	.74

	Minimum hoop reinforcement in square inches per linear foot of barrel											
Internal diameter of pipe in inches	57						60					
Type of reinforcement	Circular				Elliptical		Circular				Elliptical	
Nominal wall thickness, inches	4¾		5¾		4¾	5¾	5		6		5	6
Layers of reinforcement	Inner	Outer	Inner	Outer	Single	Single	Inner	Outer	Inner	Outer	Single	Single
CLASS												
A-25	0.44	0.34	0.52	0.42	0.74	0.74	0.47	0.35	0.55	0.45	0.78	0.78
B-25	.54	.33	.56	.38	.74	.74	.56	.34	.59	.41	.78	.78
C-25	.79	.46	.62	.34	.79	.74	.82	.47	.65	.35	.82	.78
D-25	1.03	.57	.78	.41	1.03	.78	1.07	.59	.83	.44	1.07	.82

	Minimum hoop reinforcement in square inches per linear foot of barrel											
Internal diameter of pipe in inches	63						66					
Type of reinforcement	Circular				Elliptical		Circular				Elliptical	
Nominal wall thickness, inches	5¼		6¼		5¼	6¼	5½		6½		5½	6½
Layers of reinforcement	Inner	Outer	Inner	Outer	Single	Single	Inner	Outer	Inner	Outer	Single	Single
CLASS												
A-25	0.49	0.37	0.58	0.48	0.82	0.82	0.51	0.39	0.62	0.50	0.86	0.86
B-25	.58	.35	.63	.43	.82	.82	.60	.36	.66	.46	.86	.86
C-25	.85	.49	.68	.38	.85	.82	.87	.50	.72	.40	.87	.86
D-25	1.10	.61	.87	.46	1.13	.87	1.14	.64	.91	.49	1.20	.91

	Minimum hoop reinforcement in square inches per linear foot of barrel															
Internal diameter of pipe in inches	69						72						78			
Type of reinforcement	Circular				Elliptical		Circular				Elliptical		Circular			
Nominal wall thickness, inches	5¾		6¾		5¾	6¾	6		7		6	7	6½		7½	
Layers of reinforcement	Inner	Outer	Inner	Outer	Single	Single	Inner	Outer	Inner	Outer	Single	Single	Inner	Outer	Inner	Outer
CLASS																
A-25	0.53	0.40	0.64	0.52	0.90	0.90	0.55	0.41	0.67	0.53	0.98	0.98	0.59	0.43	0.72	0.58
B-25	.62	.38	.69	.47	.90	.90	.68	.42	.71	.49	.98	.98	.72	.44	.77	.53
C-25	.89	.51	.74	.42	.91	.90	.98	.56	.80	.45	.98	.98	1.03	.59	.86	.48
D-25	1.18	.66	.95	.51	-------	.95	1.31	.73	1.05	.57	-------	1.05	1.39	.78	1.15	.62

TABLE C-2. *Reinforcement and wall thicknesses for reinforced concrete pressure pipe for 12-inch through 96-inch diameter*—Continued

	Minimum hoop reinforcement in square inches per linear foot of barrel											
Internal diameter of pipe in inches	84				90				96			
Type of reinforcement	Circular				Circular				Circular			
Nominal wall thickness, inches	7		8		7½		8		8		8½	
Layers of reinforcement	Inner	Outer	Inner	Outer	Inner	Outer	Inner	Outer	Inner	Outer	Inner	Outer
CLASS												
A-25	0.63	0.45	0.80	0.64	0.66	0.48	0.93	0.75	0.70	0.50	0.99	0.81
B-25	0.77	.47	.85	.59	.82	.50	.99	.69	.86	.53	1.06	.74
C-25	1.08	.63	.92	.52	1.13	.66	1.06	.62	1.19	.69	1.13	.67
D-25	1.47	.82	1.22	.67	1.52	.86	1.39	.77	1.59	.90	1.46	.81

NOTE.—Designations A, B, C, and D for class of pipe denote 5, 10, 15, and 20 feet of cover respectively. The number "25" for class of pipe denotes design hydrostatic pressure head in feet measured to centerline of pipe.

C-5. *Cast-in-Place Concrete Conduits.* This section presents wall thicknesses and required reinforcement for cast-in-place circular conduits with flat bases. The details are for conduits with inside diameters ranging from 2 to 10 feet. The conduits have been designed for loads resulting from fills whose surfaces are from 20 to 50 feet above the top of the conduits, and for the interior hydrostatic pressures which could exist in conduits in combination with fill loads. Details for intermediate conduit diameters and fill heights not shown in the tables can be obtained by interpolation. The thickness of the conduit wall should be interpolated between values shown in the tables by assuming a straight line variation for both fill height and diameter. The reinforcement shown for the next larger conduit diameter in combination with the next higher fill loading should be used.

The conduits have been designed and tables prepared for two conditions of loading. Details given in table C-3 are based on the assumption of no internal pressures inside the conduits, and the conduits have been designed to resist fill loads only. Details given in table C-4 are based on the assumption that hydrostatic pressure will exist inside the conduits, and the conduits have been designed to resist both internal pressure and external fill loads. The internal hydrostatic pressure was assumed to be uniform and to be produced by a head of water equal to the height of fill above the centerline of the conduit.

Each condition of loading was investigated for two assumed exterior-fill pressure distributions. The "normal condition" of fill pressure distribution considers a uniform vertical load on the top of the conduit and a uniform foundation reaction on the bottom, in combination with a uniform horizontal load equal to one-third of the vertical load. The vertical pressure was assumed to be equal to the product of the height of fill above the centerline of the conduit, multiplied by the unit weight of fill. (See sec. 235(d).) For the "unusual condition" of fill pressure distribution, the vertical fill pressure was left unchanged and the horizontal fill pressure was assumed equal to zero. The factors of safety used in design for the unusual fill load distribution, were reduced. The saturated weight of the fill was taken as 130 pounds per cubic foot in all cases.

Moments, thrusts, and shears for use in the design of the conduit were determined, using coefficients obtained from Beggs deformeter stress analysis [9]. The conduit was designed on the basis of Bulletin ACI 318-56 [10], with certain exceptions. The concrete was assumed to have a 28-day strength of 2,500 pounds per square inch, and the reinforcing steel was assumed to have a yield strength of 40,000 pounds per square inch. The thickness of the conduit wall was selected so that the concrete would be sufficient to resist the shearing stresses without the use of stirrups. Shear and bond stresses were computed by standard formulas [10]. An allowable shearing stress of 75 pounds per square inch was used for the normal fill load distribution, and a shearing stress of 100 pounds per square inch was permitted for the unusual fill load distribution. An allowable bond stress of 250 pounds per square inch was used for the normal fill load distribution, and a bond stress

TABLE C–3.—*Standardized design for cast-in-place conduits with external loads only*

Conduit diameter, feet	Height of fill above top of conduit, feet	Required conduit wall thickness, inches	Transverse reinforcement				Longitudinal reinforcement spaced at 18 inches ±
			Hoop reinforcement per foot of conduit length	Additional reinforcement required per foot of length throughout—			
				Sec. 1	Sec. 2	Sec. 3	
	50	28	2 No. 5 e.f.	1 No. 7 i.f.		1 No. 8 i.f.	No. 7 e.f.
10	40	22	2 No. 5 e.f.	1 No. 7 i.f.		1 No. 8 i.f.	No. 6 e.f.
	30	17	2 No. 5 e.f.	1 No. 7 i.f.	1 No. 4 o.f.	1 No. 7 i.f.	No. 5 e.f.
	20	12	2 No. 5 e.f.	1 No. 6 i.f.	1 No. 7 o.f.	1 No. 7 i.f.	No. 5 e.f.
	50	23	2 No. 5 e.f.	1 No. 5 i.f.		1 No. 7 i.f.	No. 6 e.f.
8	40	18	2 No. 5 e.f.	1 No. 5 i.f.		1 No. 6 i.f.	No. 6 e.f.
	30	14	2 No. 5 e.f.	1 No. 4 i.f.		1 No. 6 i.f.	No. 5 e.f.
	20	10	2 No. 5 e.f.	1 No. 4 i.f.	1 No. 4 o.f.	1 No. 6 i.f.	No. 4 e.f.
	50	18	2 No. 5 e.f.	1 No. 5 i.f.		1 No. 5 i.f.	No. 6 e.f.
6	40	14	2 No. 5 e.f.	1 No. 4 i.f.		1 No. 4 i.f.	No. 5 e.f.
	30	11	2 No. 5 e.f.	1 No. 4 i.f.		1 No. 4 i.f.	No. 4 e.f.
	20	8	2 No. 5 e.f.			1 No. 4 i.f.	No. 4 e.f.
	50	13	2 No. 5 e.f.				No. 5 e.f.
4	40	10	2 No. 5 e.f.				No. 4 e.f.
	30	8	2 No. 5 e.f.				No. 4 e.f.
	20	7	2 No. 5 c.s.				No. 5 c.s.
	50	8	2 No. 5 e.f.				No. 4 e.f.
2	40	8	2 No. 5 e.f.				No. 4 e.f.
	30	6	2 No. 5 c.s.				No. 5 c.s.
	20	6	2 No. 5 c.s.				No. 5 c.s.

i.f. denotes inside face, o.f. denotes outside face, and e.f. denotes each face for conduits having 2 layers of reinforcement. c.s. denotes center of section for conduits having 1 layer of reinforcement.

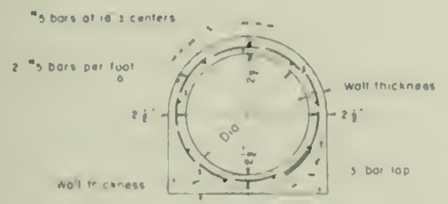
TABLE C–4.—*Standardized design for cast-in-place conduits with external loads and internal pressures*

Conduit diameter, feet	Height of fill above top of conduit, feet	Required conduit wall thickness, inches	Transverse reinforcement				Longitudinal reinforcement spaced at 18 inches ±
			Hoop reinforcement per foot of conduit length	Additional reinforcement required per foot of length throughout—			
				Sec. 1	Sec. 2	Sec. 3	
	50	28	2 No. 5 e.f.	1 No. 9 i.f.		1 No. 10 i.f.	No. 7 e.f.
10	40	22	2 No. 5 e.f.	1 No. 8 i.f.	1 No. 4 o.f.	1 No. 9 i.f.	No. 6 e.f.
	30	17	2 No. 5 e.f.	1 No. 8 i.f.	1 No. 6 o.f.	1 No. 9 i.f.	No. 5 e.f.
	20	12	2 No. 5 e.f.	1 No. 7 i.f.	1 No. 7 o.f.	1 No. 9 i.f.	No. 5 e.f.
	50	23	2 No. 5 e.f.	1 No. 7 i.f.		1 No. 8 i.f.	No. 6 e.f.
8	40	18	2 No. 5 e.f.	1 No. 6 i.f.		1 No. 8 i.f.	No. 6 e.f.
	30	14	2 No. 5 e.f.	1 No. 6 i.f.	1 No. 5 o.f.	1 No. 7 i.f.	No. 5 e.f.
	20	10	2 No. 5 e.f.	1 No. 6 i.f.	1 No. 6 o.f.	1 No. 7 i.f.	No. 4 e.f.
	50	18	2 No. 5 e.f.	1 No. 5 i.f.		1 No. 6 i.f.	No. 6 e.f.
6	40	14	2 No. 5 e.f.	1 No. 5 i.f.		1 No. 6 i.f.	No. 5 e.f.
	30	11	2 No. 5 e.f.	1 No. 4 i.f.		1 No. 5 i.f.	No. 4 e.f.
	20	8	2 No. 5 e.f.	1 No. 4 i.f.	1 No. 4 o.f.	1 No. 5 i.f.	No. 4 e.f.
	50	13	2 No. 5 e.f.	1 No. 4 i.f.		1 No. 5 i.f.	No. 5 e.f.
4	40	10	2 No. 5 e.f.			1 No. 4 i.f.	No. 4 e.f.
	30	8	2 No. 5 e.f.				No. 4 e.f.
	20	7	2 No. 5 c.s.				No. 5 c.s.
	50	8	2 No. 5 e.f.				No. 4 e.f.
2	40	8	2 No. 5 e.f.				No. 4 e.f.
	30	6	2 No. 5 c.s.				No. 5 c.s.
	20	6	2 No. 5 c.s.				No. 5 c.s.

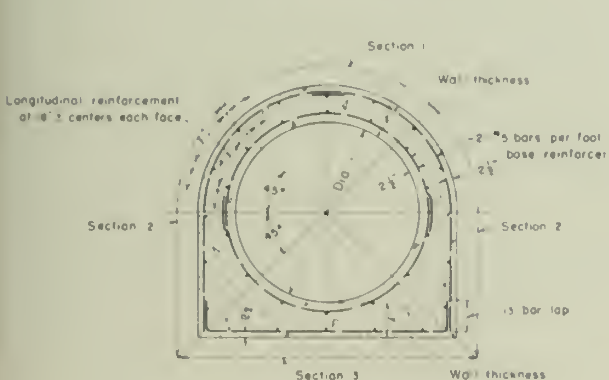
i.f. denotes inside face, o.f. denotes outside face, and e.f. denotes each face for conduits having 2 layers of reinforcement. c.s. denotes center of section for conduits having 1 layer of reinforcement.

of 333 pounds per square inch was permitted for the unusual fill load distribution.

The ultimate strength method of design [11, 12] was used to determine the required areas of reinforcement to resist moments and thrusts. A factor of safety of 2 was used in designing for the normal fill load distribution, and a factor of safety of 1.5 was used in designing for the unusual fill load distribution. In almost every instance, the design of the conduit was governed by the unusual fill load distribution. Typical reinforcement patterns for the circular conduit are shown in figure C–4.



(A) TYPICAL PATTERN WHERE CONDUIT WITH ONE LAYER OF STEEL IS REQUIRED



(B) TYPICAL PATTERN WHERE CONDUIT WITH TWO LAYERS OF STEEL ARE REQUIRED

Figure C–4. **Typical reinforcement patterns for circular cast-in-place concrete conduits.**

C–6. *Bibliography.*

[1] Terzaghi, Karl, "Theoretical Soil Mechanics," John Wiley & Sons, Inc., New York, N.Y., 1943.

[2] Terzaghi, Karl, and Peck, R. B., "Soil Mechanics in Engineering Practices," John Wiley & Sons, Inc., New York, N.Y., 1948.

[3] Tschebotarioff, G. P., "Soil Mechanics, Foundations, and Earth Structures," McGraw-Hill Book Co., Inc., New York, N.Y., 1951.

[4] Huntington, W. C., "Earth Pressures and Retaining Walls," John Wiley & Sons, Inc., New York, N.Y., 1957.

[5] "Small Retaining Walls—Construction and Tables," Structural and Railways Bureau, Portland Cement Association, Chicago, Ill.

[6] "Draft for a Revision of Part 29 of the Boston Building Code," Journal of the Boston Society of Civil Engineers, vol. 45, January 1958, p. 49.

[7] Gibbs, H. J., and Holtz, W. G., "Research on Determining the Density of Sands by Spoon Penetration Testing," Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, London, 1957, vol. 1.

[8] Olander, H. C., Stress Analysis of Concrete Pipe, Engineering Monograph No. 6, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colo., October 1950.

[9] Phillips, H. B., "Beggs Deformeter Stress Analysis of Single-Barrel Conduits," Engineering Monograph No. 14, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colo., April 1952.

[10] "Building Code Requirements for Reinforced Concrete" (ACI 318-56), American Concrete Institute, Detroit, Mich.

[11] Whitney, C. S., and Cohen, Edward, "Guide for Ultimate Strength Design of Reinforced Concrete," Journal ACI, vol. 28, November 1956. (Reprints available from American Concrete Institute, 18263 W. McNichols Road, Detroit 19, Mich.)

[12] "Ultimate-Strength Theory Simplifies Combined-Bending and Axial-Load Design," Structural and Railways Bureau, Portland Cement Association, 1955.

Soil Mechanics Nomenclature

DR. J. W. HILF [1]

D–1. *Introduction.* The following definitions of terms and symbols were selected from a draft of ASTM Designation D 653–57, "Standard Definitions of Terms and Symbols Relating to Soil Mechanics," prepared by Subcommittee G–3, Nomenclature and Definitions, of ASTM Committee D–18, Soil for Engineering Purposes, in cooperation with the Committee on Glossary of Terms and Definitions in Soil Mechanics, Soil Mechanics and Foundations Division, American Society of Civil Engineers. The list that follows is an abbreviated version of the ASTM designation, in which most of the cross-references and terms which have little or no relation to the subject matter of this text were omitted.

Units, where applicable, are indicated in capital letters on the right-hand side, under the item, and immediately above the definition. The letters denote:

F, force, such as pound, ton, gram, kilogram.

L, length, such as inch, foot, centimeter.

T, time, such as second, minute.

D, dimensionless.

Positive exponents designate multiples in the numerator.

Negative exponents designate multiples in the denominator.

Degrees of angles are indicated as "degrees" (°).

Expressing the unit either in the metric (C.G.S.) or English system has been purposely omitted in order to leave the choice of the system and specific unit to the engineer and the particular application. For example, FL^{-2} may be expressed in pounds per square inch, kilograms per square centimeter, tons per square foot, etc.; LT^{-1} may be expressed in feet per minute, centimeters per second, etc. No significance should be placed on the order in which symbols are presented where two or more are given.

The following letters of the Greek alphabet are used in this nomenclature:

Greek letter	*Greek name*
α	Alpha
β	Beta
γ	Gamma
Δ, δ	Delta
ϵ	Epsilon
θ	Theta
μ	Mu
σ	Sigma
τ	Tau
ϕ	Phi
ψ	Psi

D–2. *Definitions, Symbols, and Units.*

ABSORBED WATER:

Water held mechanically in a soil mass and having physical properties not substantially different from ordinary water at the same temperature and pressure.

ADHESION:

Unit: c_a — FL^{-2}

Total: C_a — F or FL^{-1}

Shearing resistance between soil and another material under zero externally applied pressure.

ADSORBED WATER:

Water in a soil mass, held by physicochemical forces, having physical properties substantially different from absorbed water or chemically combined water, at the same temperature and pressure.

[1] Engineer, Earth Dams Section, Bureau of Reclamation.

AEOLIAN DEPOSITS:

Wind-deposited material such as dune sands and loess deposits.

AIRSPACE RATIO:

G_a D

Ratio of (1) volume of water that can be drained from a saturated soil under the action of force of gravity to (2) total volume of voids.

AIR-VOID RATIO:

G_v D

The ratio of (1) the volume of airspace to (2) the total volume of voids in a soil mass.

ALLOWABLE BEARING VALUE (ALLOWABLE SOIL PRESSURE):

q_a, p_a FL^{-2}

The maximum pressure that can be permitted on foundation soil, giving consideration to all pertinent factors, with adequate safety against rupture of the soil mass or movement of the foundation of such magnitude that the structure is impaired.

ALLOWABLE PILE BEARING LOAD:

Q_a, P_a F

The maximum load that can be permitted on a pile with adequate safety against movement of such magnitude that the structure is endangered.

ALLUVIUM:

Soil the constituents of which have been transported in suspension by flowing water and subsequently deposited by sedimentation.

ANGLE OF EXTERNAL FRICTION (ANGLE OF WALL FRICTION):

δ Degrees (°)

Angle between the abscissa and the tangent of the curve representing the relationship of shearing resistance to normal stress acting between soil and surface of another material.

ANGLE OF INTERNAL FRICTION:

ϕ Degrees (°)

Angle between the abscissa and the tangent of the curve representing the relationship of shearing resistance to normal stress acting within a soil.

ANGLE OF OBLIQUITY:

α, β, θ, ψ Degrees (°)

The angle between the direction of the resultant stress or force acting on a given plane and the normal to that plane.

ANGLE OF REPOSE:

α Degrees (°)

Angle between the horizontal and the maximum slope that a soil assumes through natural processes. For dry granular soils the effect of the height of slope is negligible; for cohesive soils the effect of height of slope is so great that the angle of repose is meaningless.

ANISOTROPIC MASS:

A mass having different properties in different directions at any given point.

AQUIFER:

A water-bearing formation that provides a ground-water reservoir.

ARCHING:

The transfer of stress from a yielding part of a soil mass to adjoining less-yielding or restrained parts of the mass.

AREA OF INFLUENCE OF A WELL:

a L^2

Area surrounding a well within which the piezometric surface has been lowered when pumping has produced the maximum steady rate of flow.

AREA RATIO OF A SAMPLING SPOON, SAMPLER, OR SAMPLING TUBE:

A_r D

$A_r=\frac{D_e^2-D_i^2}{D_i^2}\times100$ where D_e represents the maximum external diameter of the sampling spoon and D_i represents the minimum internal diameter of the sampling spoon at the cutting edge. The area ratio is an indication of the volume of soil displaced by the sampling spoon (tube).

BASE COURSE (BASE):

A layer of specified or selected material of planned thickness constructed on the subgrade or subbase for the purpose of serving one or more functions such as distributing load, providing drainage, minimizing frost action, etc.

BASE EXCHANGE:

The physicochemical process whereby one species of ions adsorbed on soil particles is replaced by another species.

BEARING CAPACITY (OF A PILE):

Q_p, P_p F

The load per pile required to produce a condition of failure.

BEDROCK (LEDGE):

Rock of relatively great thickness and extent in its native location.

BENTONITIC CLAY:

A clay with a high content of the mineral montmorillonite, usually characterized by high swelling on wetting.

BERM:

A shelf that breaks the continuity of a slope.

BINDER (SOIL BINDER):

Portion of soil passing No. 40 United States standard sieve.

BOULDER:

A rock fragment, usually rounded by weathering or abrasion, with an average dimension of 12 inches or more.

BOULDER CLAY:

A geological term used to designate glacial drift that has not been subjected to the sorting action of water and therefore contains particles from boulders to clay sizes.

BULKING:

The increase in volume of a material due to manipulation. Rock bulks upon being excavated; damp sand bulks if loosely deposited, as by dumping, because the "apparent cohesion" prevents movement of the soil particles to form a reduced volume.

CALIFORNIA BEARING RATIO:

CBR D

The ratio of (1) the force per unit area required to penetrate a soil mass with a 3-square-inch circular piston (approximately 2-inch-diameter) at the rate of 0.05 inch per minute to (2) that required for corresponding penetration of a standard material. The ratio is usually determined at 0.1-inch penetration, although other penetrations are sometimes used. Original California procedures required determination of the ratio at 0.1-inch intervals to 0.5 inch. Corps of Engineers' procedures require determination of the ratio at 0.1 inch and 0.2 inch. Where the ratio at 0.2 inch is consistently higher than at 0.1 inch, the ratio at 0.2 inch is used.

CAPILLARY ACTION (CAPILLARITY):

The rise or movement of water in the interstices of a soil due to capillary forces.

CAPILLARY FRINGE ZONE:

The zone above the free water elevation in which water is held by capillary action.

CAPILLARY HEAD:

h L

The potential, expressed in head of water, that causes the water to flow by capillary action.

CAPILLARY MIGRATION (CAPILLARY FLOW):

The movement of water by capillary action.

CAPILLARY RISE (HEIGHT OF CAPILLARY RISE):

h_c L

The height above a free water elevation to which water will rise by capillary action.

CAPILLARY WATER:

Water subject to the influence of capillary action.

CLAY (CLAY SOIL):

Fine-grained soil or the fine-grained portion of soil that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and which exhibits considerable strength when air-dry. The term has been used to designate the percentage finer than 0.002 mm. (0.005 in some cases), but it is strongly recommended that this usage be discontinued, since there is ample evidence that from an engineering standpoint the properties described in the above definition are many times more important.

CLAY SIZE:

That portion of the soil finer than 0.002 mm. (0.005 mm. in some cases). (See discussion under Clay.)

COBBLE (COBBLESTONE):

A rock fragment usually rounded or semi-rounded with an average dimension between 3 and 12 inches.

COEFFICIENT OF COMPRESSIBILITY (COEFFICIENT OF COMPRESSION):

a_v L^2F^{-1}

The secant slope, for a given pressure increment, of the pressure-void ratio curve. Where a stress-strain curve is used, the slope of this curve is equal to $\frac{(a_v)}{1+e}$.

COEFFICIENT OF CONSOLIDATION:

c_v L^2T^{-1}

A coefficient utilized in the theory of consolidation, containing the physical constants of a soil affecting its rate of volume change.

$c_v = \frac{k(1+e)}{a_v \cdot \gamma_w}$, wherein

k = coefficient of permeability, LT^{-1}

e = void ratio, D

a_v = coefficient of compressibility, L^2F^{-1}

γ_w = unit weight of water, FL^{-3}

NOTE.—In the literature published prior to 1935, the coefficient of consolidation, usually designated c, was defined by the equation $c = \frac{k}{a_v \cdot \gamma_w (1+e)}$. This original definition of the coefficient of consolidation may be found in some more recent papers and care should be taken to avoid confusion.

COEFFICIENT OF EARTH PRESSURE:

K D

The principal stress ratio at a point in a soil mass.

ACTIVE:

K_A D

The minimum ratio of (1) the minor principal stress to (2) the major principal stress. This is applicable where the soil has yielded sufficiently to develop a lower limiting value of the minor principal stress.

AT REST:

K_o D

The ratio of (1) the minor principal stress to (2) the major principal stress. This is applicable where the soil mass is in its natural state without having been permitted to yield or without having been compressed.

PASSIVE:

K_p D

The maximum ratio of (1) the major principal stress to (2) the minor principal stress. This is applicable where the soil has been compressed sufficiently to develop an upper limiting value of the major principal stress.

COEFFICIENT OF INTERNAL FRICTION:

The tangent of the angle of internal friction. (See Internal Friction.)

COEFFICIENT OF PERMEABILITY (PERMEABILITY):

k LT^{-1}

The rate of discharge of water under laminar flow conditions through a unit cross-sectional area of a porous medium under a unit hydraulic gradient and standard temperature conditions (usually 20° C.).

COEFFICIENT OF SUBGRADE REACTION (MODULUS OF SUBGRADE REACTION):

k FL^{-3}

Ratio of (1) load per unit area of horizontal surface of a mass of soil to (2) corresponding settlement of the surface. It is determined as the slope of the secant, drawn between the point corresponding to zero settlement and the point of 0.05-inch settlement, of a load settlement curve obtained from a plate load test on a soil using a 30-inch or greater diameter loading plate. It is used in the design of concrete pavements by the Westergaard method.

COEFFICIENT OF UNIFORMITY:

C_u D

The ratio D_{60}/D_{10}, where D_{60} is the particle diameter corresponding to 60 percent finer on the grain-size curve, and D_{10} is the particle diameter corresponding to 10 percent finer on the grain-size curve.

COEFFICIENT OF VISCOSITY (COEFFICIENT OF ABSOLUTE VISCOSITY):

μ FTL^{-2}

The shearing force per unit area required to maintain a unit difference in velocity between two parallel layers of a fluid a unit distance apart.

COEFFICIENT OF VOLUME COMPRESSIBILITY (MODULUS OF VOLUME CHANGE):

m_v L^2F^{-1}

The compression of a soil layer per unit of original thickness due to a given unit increase in pres-

sure. It is numerically equal to the coefficient of compressibility divided by one plus the original void ratio: $\frac{a_v}{1+e}$

COHESION:

The portion of the shear strength of a soil indicated by the term c in Coulomb's equation, $s=c+\bar{\sigma}\tan\phi$.

APPARENT COHESION:

Cohesion in granular soils due to capillary forces.

COHESIONLESS SOIL:

A soil that when unconfined has little or no strength when air-dried, and that has little or no cohesion when submerged.

COHESIVE SOIL:

A soil that when unconfined has considerable strength when air-dried, and that has significant cohesion when submerged.

COLLOIDAL PARTICLES:

Soil particles that are so small that the surface activity has an appreciable influence on the properties of the aggregate.

COMPACTION:

The densification of a soil by means of mechanical manipulation.

COMPACTION CURVE (PROCTOR CURVE) (MOISTURE-DENSITY CURVE):

The curve showing the relationship between the dry unit weight (density) and the water content of a soil for a given compactive effort.

COMPACTION TEST (MOISTURE-DENSITY TEST):

A laboratory compacting procedure whereby a soil at a known water content is placed in a specified manner into a mold of given dimensions, subjected to a compactive effort of controlled magnitude, and the resulting unit weight determined. The procedure is repeated for various water contents sufficient to establish a relation between water content and unit weight.

COMPRESSIBILITY:

Property of a soil pertaining to its susceptibility to decrease in volume when subjected to load.

COMPRESSION INDEX:

C_c D

The slope of the linear portion of the pressure-void ratio curve on a semilog plot.

COMPRESSIVE STRENGTH (UNCONFINED COMPRESSIVE STRENGTH):

p_c, q_u FL^{-2}

The load per unit area at which an unconfined prismatic or cylindrical specimen of soil will fail in a simple compression test.

CONCENTRATION FACTOR:

n D

A parameter used in modifying the Boussinesq equations to describe various distributions of vertical stress.

CONSISTENCY:

The relative ease with which a soil can be deformed.

CONSISTENCY INDEX:

See Relative Consistency.

CONSOLIDATED DRAINED TEST (SLOW TEST):

A soil test in which essentially complete consolidation under the confining pressure is followed by additional axial (or shearing) stress applied in such a manner that even a fully saturated soil of low permeability can adapt itself completely (fully consolidate) to the changes in stress due to the additional axial (or shearing) stress.

CONSOLIDATED UNDRAINED TEST (CONSOLIDATED QUICK TEST):

A test in which complete consolidation under the vertical load (in a direct shear test) or under the confining pressure (in a triaxial test) is followed by a shear at constant water content.

CONSOLIDATION:

The gradual reduction in volume of a soil mass resulting from an increase in compressive stress.

INITIAL CONSOLIDATION (INITIAL COMPRESSION):

A comparatively sudden reduction in volume of a soil mass under an applied load due principally to expulsion and compression of gas in the soil voids preceding primary consolidation.

PRIMARY CONSOLIDATION (PRIMARY COMPRESSION) (PRIMARY TIME EFFECT):

The reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to a squeezing out of water from the void spaces of the mass and accompanied by a transfer of the load from the soil water to the soil solids.

SECONDARY CONSOLIDATION (SECONDARY COMPRESSION) (SECONDARY TIME EFFECT):

The reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to the adjustment of the internal structure of the soil mass after most of the load has been transferred from the soil water to the soil solids.

CONSOLIDATION RATIO:

U_z D

The ratio of (1) the amount of consolidation at a given distance from a drainage surface and at a given time to (2) the total amount of consolidation obtainable at that point under a given stress increment.

CONSOLIDATION TEST:

A test in which the specimen is laterally confined in a ring and is compressed between porous plates.

CONSOLIDATION-TIME CURVE (TIME CURVE) (CONSOLIDATION CURVE) (THEORETICAL TIME CURVE):

A curve that shows the relation between (1) the degree of consolidation and (2) the elapsed time after the application of a given increment of load.

CREEP:

Slow movement of rock debris or soil usually imperceptible except to observations of long duration.

CRITICAL CIRCLE (CRITICAL SURFACE):

The sliding surface assumed in a theoretical analysis of a soil mass for which the factor of safety is a minimum.

CRITICAL DENSITY:

The unit weight of a saturated granular material below which it will lose strength and above which it will gain strength when subjected to rapid deformation. The critical density of a given material is dependent on many factors.

CRITICAL HEIGHT:

H_c L

The maximum height at which a vertical or sloped bank of soil will stand unsupported under a given set of conditions.

CRITICAL SLOPE:

The maximum angle with the horizontal at which a sloped bank of soil of given height will stand unsupported.

DEFLOCCULATING AGENT (DEFLOCCULANT) (DISPERSING AGENT):

An agent that prevents fine soil particles in suspension from coalescing to form flocs.

DEFORMATION:

Change in shape.

DEGREE OF CONSOLIDATION (PERCENT CONSOLIDATION):

U D

The ratio, expressed as a percentage, of (1) the amount of consolidation at a given time within a soil mass, to (2) the total amount of consolidation obtainable under a given stress condition.

DENSITY:

See Unit Weight.

NOTE.—Although it is recognized that density is defined as mass per unit volume, in the field of soil mechanics the term is frequently used in place of unit weight.

DEVIATOR STRESS:

$\triangle, \sigma$ FL^{-2}

The difference between the major and minor principal stresses in a triaxial test.

DILATANCY:

The expansion of cohesionless soils when subject to shearing deformation.

DIRECT SHEAR TEST:

A shear test in which soil under an applied normal load is stressed to failure by moving one section of the soil container (shear box) relative to the other section.

DISCHARGE VELOCITY:

v LT^{-1}

Rate of discharge of water through a porous medium per unit of total area perpendicular to the direction of flow.

DRAWDOWN:

L

Vertical distance the free water elevation is lowered or the reduction of the pressure head due to the removal of free water.

EARTH PRESSURE:

Unit: p — FL^{-2}

Total: P — F or FL^{-1}

The pressure or force exerted by soil on any boundary.

ACTIVE EARTH PRESSURE:

P_A, p_A

The minimum value of earth pressure. This condition exists when a soil mass is permitted to yield sufficiently to cause its internal shearing resistance along a potential failure surface to be completely mobilized.

AT REST:

P_O, p_O

The value of the earth pressure when the soil mass is in its natural state without having been permitted to yield or without having been compressed.

PASSIVE EARTH PRESSURE:

P_P, p_P

The maximum value of earth pressure. This condition exists when a soil mass is compressed sufficiently to cause its internal shearing resistance along a potential failure surface to be completely mobilized.

EFFECTIVE DIAMETER (EFFECTIVE SIZE):

D_{10}, D_e — L

Particle diameter corresponding to 10 percent finer on the grain-size curve.

EFFECTIVE FORCE:

$\overline{F}$ — F

The force transmitted through a soil mass by intergranular pressures.

EFFECTIVE POROSITY (EFFECTIVE DRAINAGE POROSITY):

n_e — D

The ratio of (1) the volume of the voids of a soil mass that can be drained by gravity to (2) the total volume of the mass.

ELASTIC STATE OF EQUILIBRIUM:

State of stress within a soil mass when the internal resistance of the mass is not fully mobilized.

EQUIPOTENTIAL LINE:

Line along which water will rise to the same elevation in piezometric tubes.

EQUIVALENT DIAMETER (EQUIVALENT SIZE):

D — L

The diameter of a hypothetical sphere composed of material having the same specific gravity as that of the actual soil particle and of such size that it will settle in a given liquid at the same terminal velocity as the actual soil particle.

EQUIVALENT FLUID:

A hypothetical fluid having a unit weight such that it will produce a pressure against a lateral support presumed to be equivalent to that produced by the actual soil. This simplified approach is valid only when deformation conditions are such that the pressure increases linearly with depth and the wall friction is neglected.

EXCHANGE CAPACITY:

The capacity to exchange ions as measured by the quantity of exchangeable ions in a soil.

FILL:

Manmade deposits of natural soils and waste materials.

FILTER (PROTECTIVE FILTER):

A layer or combination of layers of pervious materials designed and installed in such a manner as to provide drainage, yet prevent the movement of soil particles due to flowing water.

FINES:

Portion of a soil finer than a No. 200 United States standard sieve.

FLOC:

Loose, open-structured mass formed in a suspension by the aggregation of minute particles.

FLOCCULATION:

The process of forming flocs.

FLOW CHANNEL:

The portion of a flow net bounded by two adjacent flow lines.

FLOW CURVE:

The locus of points obtained from a standard liquid limit test and plotted on a graph representing water content as ordinate on an arithmetic scale and the number of blows as abscissa on a logarithmic scale.

FLOW FAILURE:

Failure in which a soil mass moves over relatively long distances in a fluidlike manner.

FLOW INDEX:

F_w, I_f D

The slope of the flow curve obtained from a liquid limit test, expressed as the difference in water contents at 10 blows and at 100 blows.

FLOW LINE:

The path that a particle of water follows in its course of seepage under laminar flow conditions.

FLOW NET:

A graphical representation of flow lines and equipotential lines used in the study of seepage phenomena.

FLOW SLIDE:

The failure of a sloped bank of soil in which the movement of the soil mass does not take place along a well-defined surface of sliding.

FLOW VALUE:

N_ϕ D

A quantity equal to $\tan^2\left(45°+\frac{\phi}{2}\right)$.

FOOTING:

Portion of the foundation of a structure that transmits loads directly to the soil.

FOUNDATION:

Lower part of a structure that transmits the load to the earth.

FOUNDATION SOIL:

Upper part of the earth mass carrying the load of the structure.

FREE WATER (GRAVITATIONAL WATER) (GROUND WATER) (PHREATIC WATER):

Water that is free to move through a soil mass under the influence of gravity.

FREE WATER ELEVATION (WATER TABLE) (GROUND-WATER SURFACE) (FREE WATER SURFACE) (GROUND-WATER ELEVATION):

Elevations at which the pressure in the water is zero with respect to the atmospheric pressure.

FROST ACTION:

Freezing and thawing of moisture in materials and the resultant effects on these materials and on structures of which they are a part or with which they are in contact.

FROST BOIL:

(1) Softening of soil occurring during a thawing period due to the liberation of water from ice lenses or layers.

(2) Breaking of a highway or airfield pavement under traffic and the ejection of subgrade soil in a soft and soupy condition caused by the melting of ice lenses formed by frost action.

FROST HEAVE:

The raising of a surface due to the accumulation of ice in the underlying soil.

GLACIAL TILL (TILL):

Material deposited by glaciation, usually composed of a wide range of particle sizes, which has not been subjected to the sorting action of water.

GRADATION (GRAIN SIZE DISTRIBUTION) (SOIL TEXTURE):

Proportion of material of each grain size present in a given soil.

GRAIN SIZE ANALYSIS (MECHANICAL ANALYSIS):

The process of determining gradation.

GRAVEL:

Rounded or semirounded particles of rock that will pass a 3-inch and be retained on a No. 4 United States standard sieve.

HARDPAN:

Layer of extremely dense soil.

HEAVE:

Upward movement of soil caused by expansion or displacement resulting from phenomena such as: moisture absorption, removal of overburden, driving of piles, and frost action.

HOMOGENEOUS MASS:

A mass that exhibits essentially the same physical properties at every point throughout the mass.

HORIZON (SOIL HORIZON):

One of the layers of the soil profile, distinguished principally by its texture, color, structure, and chemical content.

A HORIZON:

The uppermost layer of a soil profile from which inorganic colloids and other soluble materials have been leached. Usually contains remnants of organic life.

B HORIZON:

The layer of a soil profile in which material leached from the overlying A horizon is accumulated.

C HORIZON:

Undisturbed parent material from which the overlying soil profile has been developed.

HUMUS:

A brown or black material formed by the partial decomposition of vegetable or animal matter; the organic portion of soil.

HYDRAULIC GRADIENT:

i, s D

The loss of hydraulic head per unit distance of flow; $\frac{dh}{dL}$

CRITICAL HYDRAULIC GRADIENT:

i_c D

Hydraulic gradient at which the intergranular pressure in a mass of cohesionless soil is reduced to zero by the upward flow of water.

HYDROSTATIC PRESSURE:

u_o FL^{-2}

The pressure in a liquid under static conditions; the product of the unit weight of the liquid and the difference in elevation between the given point and the free water elevation.

EXCESS HYDROSTATIC PRESSURE (HYDROSTATIC EXCESS PRESSURE):

$\overline{u}$, u FL^{-2}

The pressure that exists in pore water in excess of the hydrostatic pressure.

HYGROSCOPIC CAPACITY (HYGROSCOPIC COEFFICIENT):

w_c D

Ratio of (1) the weight of water absorbed by a dry soil in a saturated atmosphere at a given temperature to (2) the weight of the oven-dried soil.

HYGROSCOPIC WATER CONTENT:

w_H D

The water content of an air-dried soil.

INTERNAL FRICTION:

FL^{-2}

The portion of the shearing strength of a soil indicated by the terms $p \tan \phi$ in Coulomb's equation $s = c + p \tan \phi$. It is usually considered to be due to the interlocking of the soil grains and the resistance to sliding between the grains.

ISOCHRONE:

A curve showing the distribution of the excess hydrostatic pressure at a given time during a process of consolidation.

ISOTROPIC MASS:

A mass having the same property (or properties) in all directions.

KAOLIN:

A variety of clay containing a high percentage of kaolinite.

LAMINAR FLOW (STREAMLINE FLOW) (VISCOUS FLOW):

Flow in which each water particle moves in a direction parallel to every other particle, and in which the head loss is proportional to the first power of the velocity.

LANDSLIDE (SLIDE):

The failure of a sloped bank of soil in which the movement of the soil mass takes place along a surface of sliding.

LEACHING:

The removal of soluble soil material and colloids by percolating water.

LINE OF CREEP (PATH OF PERCOLATION):

The path that water follows along the surface of contact between the foundation soil and the base of a dam or other structure.

LINE OF SEEPAGE (SEEPAGE LINE) (PHREATIC LINE):

The upper free water surface of the zone of seepage.

LINEAR EXPANSION:

L_E D

The increase in one dimension of a soil mass, expressed as a percentage of that dimension at the shrinkage limit, when the water content is increased from the shrinkage limit to any given water content.

LINEAR SHRINKAGE:

L_S D

Decrease in one dimension of a soil mass, expressed as a percentage of the original dimension,

when the water content is reduced from a given value to the shrinkage limit.

LIQUID LIMIT:

LL, L_w, w_L D

(1) The water content corresponding to the arbitrary limit between the liquid and plastic states of consistency of a soil.

(2) The water content at which a pat of soil, cut by a groove of standard dimensions, will flow together for a distance of one-half inch under the impact of 25 blows in a standard liquid limit apparatus.

LIQUEFACTION (SPONTANEOUS LIQUEFACTION):

The sudden large decrease of the shearing resistance of a cohesionless soil. It is caused by a collapse of the structure by shock or other type of strain and is associated with a sudden but temporary increase of the pore-fluid pressure. It involves a temporary transformation of the material into a fluid mass.

LIQUIDITY INDEX (WATER PLASTICITY RATIO) (RELATIVE WATER CONTENT):

B, R_w, I_L D

The ratio, expressed as a percentage, of (1) the natural water content of a soil minus its plastic limit to (2) its plasticity index.

LOAM:

A mixture of sand, silt, or clay, or a combination of any of these, with organic matter (humus). It is sometimes called topsoil in contrast to the subsoils that contain little or no organic matter.

LOESS:

A uniform aeolian deposit of silty material having an open structure and relatively high cohesion due to cementation of clay or calcareous material at grain contacts. A characteristic of loess deposits is that they can stand with nearly vertical slopes.

MODULUS OF ELASTICITY (MODULUS OF DEFORMATION):

E, *M* FL^{-2}

The ratio of stress to strain for a material under given loading conditions; numerically equal to the slope of the tangent or the secant of a stress-strain curve. The use of the term Modulus of Elasticity is recommended for materials that deform in accordance with Hooke's law; the term Modulus of Deformation for materials that deform otherwise.

MOHR CIRCLE:

A graphical representation of the stresses acting on the various planes at a given point.

MOHR ENVELOPE (RUPTURE ENVELOPE) (RUPTURE LINE):

The envelope of a series of Mohr Circles representing stress conditions at failure for a given material. According to Mohr's rupture hypothesis, a rupture envelope is the locus of points the coordinates of which represent the combinations of normal and shearing stresses that will cause a given material to fail.

MOISTURE CONTENT (WATER CONTENT):

w D

The ratio, expressed as a percentage, of (1) the weight of water in a given soil mass to (2) the weight of solid particles.

MOISTURE EQUIVALENT:

CENTRIFUGE MOISTURE EQUIVALENT:

W_c, *CME* D

The water content of a soil after it has been saturated with water and then subjected for one hour to a force equal to 1,000 times that of gravity.

FIELD MOISTURE EQUIVALENT:

FME

The minimum water content, expressed as a percentage of the weight of the oven-dried soil, at which a drop of water placed on a smoothed surface of the soil will not immediately be absorbed by the soil but will spread out over the surface and give it a shiny appearance.

MUCK

An organic soil of very soft consistency.

MUD:

A mixture of soil and water in a fluid or weakly solid state.

MUSKEG:

Level, practically treeless areas supporting dense growth consisting primarily of grasses. The surface of the soil is covered with a layer of partially decayed grass and grass roots which is usually wet and soft when not frozen.

NORMALLY CONSOLIDATED SOIL DEPOSIT:

A soil deposit that has never been subjected to a pressure greater than the existing overburden pressure.

OPTIMUM MOISTURE CONTENT (OPTIMUM WATER CONTENT):

OMC, W_o D

The water content at which a soil can be compacted to the maximum dry unit weight by a given compactive effort.

ORGANIC CLAY:

A clay with a high organic content.

ORGANIC SILT:

A silt with a high organic content.

ORGANIC SOIL:

Soil with a high organic content. In general, organic soils are very compressible and have poor load-sustaining properties.

OVERCONSOLIDATED SOIL DEPOSIT:

A soil deposit that has been subjected to pressure greater than the present overburden pressure.

PARENT MATERIAL:

Material from which a soil has been derived.

PEAT:

A fibrous mass of organic matter in various stages of decomposition, generally dark brown to black in color and of spongy consistency.

PENETRATION RESISTANCE (STANDARD PENETRATION RESISTANCE) (PROCTOR PENETRATION RESISTANCE):

p_R, N FL^{-2} or blows L^{-1}

(1) Number of blows of a hammer of specified weight falling a given distance required to produce a given penetration into soil of a pile, casing, or sampling tube.

(2) Unit load required to maintain constant rate of penetration into soil of a probe or instrument.

(3) Unit load required to produce a specified penetration into soil at a specified rate of a probe or instrument. For a Proctor needle, the specified penetration is 2½ inches and the rate is ½ inch per second.

PENETRATION RESISTANCE CURVE (PROCTOR PENETRATION CURVE):

The curve showing the relationship between (1) the penetration resistance and (2) the water content.

PERCENT COMPACTION:

The ratio, expressed as a percentage, of (1) dry unit weight of a soil to (2) maximum unit weight obtained in a laboratory compaction test.

PERCENT SATURATION (DEGREE OF SATURATION):

S D

The ratio, expressed as a percentage, of (1) the volume of water in a given soil mass to (2) the total volume of intergranular space (voids).

PERCHED WATER TABLE:

A water table usually of limited area maintained above the normal free water elevation by the presence of an intervening relatively impervious confining stratum.

PERCOLATION:

The movement of gravitational water through soil. (See Seepage.)

PERMAFROST:

Perennially frozen soil.

pH

pH D

An index of the acidity or alkalinity of a soil in terms of the logarithm of the reciprocal of the hydrogen ion concentration.

PIEZOMETER:

An instrument for measuring pressure head.

PIEZOMETRIC SURFACE:

The surface at which water will stand in a series of piezometers.

PILE:

Relatively slender structural element which is driven, or otherwise introduced, into the soil, usually for the purpose of providing vertical or lateral support.

PIPING:

The movement of soil particles by percolating water leading to the development of channels.

PLASTIC EQUILIBRIUM:

State of stress within a soil mass or a portion thereof, which has been deformed to such an extent that its ultimate shearing resistance is mobilized.

ACTIVE STATE OF PLASTIC EQUILIBRIUM:

Plastic equilibrium obtained by an expansion of a mass.

PASSIVE STATE OF PLASTIC EQUILIBRIUM:

Plastic equilibrium obtained by a compression of a mass.

PLASTICITY:

The property of a soil which allows it to be deformed beyond the point of recovery without cracking or appreciable volume change.

PLASTIC FLOW (PLASTIC DEFORMATION):

The deformation of a plastic material beyond the point of recovery, accompanied by continuing deformation with no further increase in stress.

PLASTIC LIMIT:

w_p, PL, P_w D

(1) The water content corresponding to an arbitrary limit between the plastic and the semisolid states of consistency of a soil.

(2) Water content at which a soil will just begin to crumble when rolled into a thread approximately one-eighth inch in diameter.

PLASTIC SOIL:

A soil that exhibits plasticity.

PLASTIC STATE (PLASTIC RANGE):

The range of consistency within which a soil exhibits plastic properties.

PLASTICITY INDEX:

I_p, PI, I_w D

Numerical difference between the liquid limit and the plastic limit.

PORE PRESSURE (PORE WATER PRESSURE):

See Neutral Stress under Stress.

POROSITY:

n D

The ratio, usually expressed as a percentage, of (1) the volume of voids of a given soil mass to (2) the total volume of the soil mass.

POTENTIAL DROP:

Δh L

The difference in pressure head between two equipotential lines.

PRECONSOLIDATION PRESSURE (PRESTRESS):

p_c FL^{-2}

The greatest pressure to which a soil has been subjected.

PRESSURE:

p FL^{-2}

The load divided by the area over which it acts.

PRESSURE BULB:

The zone in a loaded soil mass bounded by an arbitrarily selected isobar of stress.

PRESSURE—VOID RATIO CURVE (COMPRESSION CURVE):

A curve representing the relationship between pressure and void ratio of a soil as obtained from a consolidation test. The curve has a characteristic shape when plotted on semilog paper with pressure on the log scale. The various parts of the curve and extensions to the parts have been designated as recompression, compression, virgin compression, expansion, rebound, and other descriptive names by various authorities.

PRINCIPAL PLANE:

Each of three mutually perpendicular planes through a point in a soil mass on which the shearing stress is zero.

INTERMEDIATE PRINCIPAL PLANE:

The plane normal to the direction of the intermediate principal stress.

MAJOR PRINCIPAL PLANE:

The plane normal to the direction of the major principal stress.

MINOR PRINCIPAL PLANE:

The plane normal to the direction of the minor principal stress.

PROCTOR COMPACTION CURVE:

See Compaction Curve.

PROCTOR PENETRATION CURVE:

See Penetration Resistance Curve.

PROCTOR PENETRATION RESISTANCE:

See Penetration Resistance.

PROGRESSIVE FAILURE:

Failure in which the ultimate shearing resistance is progressively mobilized along the failure surface.

QUICK CONDITION (QUICKSAND):

Condition in which water is flowing upward with sufficient velocity to reduce significantly the bearing capacity of the soil through a decrease in intergranular pressure.

QUICK TEST:

See Unconsolidated Undrained Test.

RADIUS OF INFLUENCE OF A WELL:

Distance from the center of the well to the closest point at which the piezometric surface is not lowered when pumping has produced the maximum steady rate of flow.

RELATIVE CONSISTENCY:

I_c, C_r D

Ratio of (1) the liquid limit minus the natural water content to (2) the plasticity index.

RELATIVE DENSITY:

D_d D

The ratio of (1) the difference between the void ratio of a cohesionless soil in the loosest state and any given void ratio to (2) the difference between its void ratios in the loosest and in the densest states.

REMOLDED SOIL:

Soil that has had its natural structure modified by manipulation.

REMOLDING INDEX:

I_R D

The ratio of (1) the modulus of deformation of a soil in the undisturbed state to (2) the modulus of deformation of the soil in the remolded state.

REMOLDING SENSITIVITY (SENSITIVITY RATIO):

S_t D

The ratio of (1) the unconfined compressive strength of an undisturbed specimen of soil to (2) the unconfined compressive strength of a specimen of the same soil after remolding at unaltered water content.

RESIDUAL SOIL:

Soil derived in place by weathering of the underlying material.

ROCK:

Natural solid mineral matter occurring in large masses or fragments.

ROCK FLOUR:

See Silt.

SAND:

Particles of rock that will pass the No. 4 United States standard sieve and be retained on the No. 200 sieve.

SAND BOIL:

The ejection of sand and water resulting from piping.

SEEPAGE (PERCOLATION):

The slow movement of gravitational water through the soil.

SEEPAGE FORCE:

J F

The force transmitted to the soil grains by seepage.

SEEPAGE VELOCITY:

v_s, v_l LT^{-1}

The rate of discharge of seepage water through a porous medium per unit area of void space perpendicular to the direction of flow.

SENSITIVITY:

The effect of remolding on the consistency of a cohesive soil.

SHAKING TEST:

A test used to indicate the presence of significant amounts of rock flour, silt, or very fine sand in a fine-grained soil. It consists of shaking a pat of wet soil, having a consistency of thick paste, in the palm of the hand; observing the surface for a glossy or livery appearance; then squeezing the pat; and observing if a rapid apparent drying and subsequent cracking of the soil occurs.

SHEAR FAILURE (FAILURE BY RUPTURE):

Failure in which movement caused by shearing stresses in a soil mass is of sufficient magnitude to destroy or seriously endanger a structure.

GENERAL SHEAR FAILURE:

Failure in which the ultimate strength of the soil is mobilized along the entire potential surface of sliding before the structure supported by the soil is impaired by excessive movement.

LOCAL SHEAR FAILURE:

Failure in which the ultimate shearing strength of the soil is mobilized only locally along the potential surface of sliding at the time the structure supported by the soil is impaired by excessive movement.

SHEAR STRENGTH:

s FL^{-2}

The maximum resistance of a soil to shearing stresses.

SHEAR STRESS (SHEARING STRESS) (TANGENTIAL STRESS):

See Stress.

SHRINKAGE INDEX:

SI D

The numerical difference between the plastic and shrinkage limits.

SHRINKAGE LIMIT:

SL D

The maximum water content at which a reduction in water content will not cause a decrease in volume of the soil mass.

SHRINKAGE RATIO:

R D

The ratio of (1) a given volume change, expressed as a percentage of the dry volume, to (2) the corresponding change in water content above the shrinkage limit, expressed as a percentage of the weight of the oven-dried soil.

SILT (INORGANIC SILT) (ROCK FLOUR):

Material passing the No. 200 United States standard sieve that is nonplastic or very slightly plastic and that exhibits little or no strength when air-dried.

SILT SIZE:

That portion of the soil finer than 0.02 mm. and coarser than 0.002 mm. (0.05 mm. and 0.005 mm. in some cases).

SKIN FRICTION:

f FL^{-2}

The frictional resistance developed between soil and a structure.

SLAKING:

The process of breaking up or sloughing when an indurated soil is immersed in water.

SLOW TEST:

See Consolidated-Drained Test.

SOIL (EARTH):

Sediments or other unconsolidated accumulations of solid particles produced by the physical and chemical disintegration of rocks, and which may or may not contain organic matter.

SOIL MECHANICS:

The application of the laws and principles of mechanics and hydraulics to engineering problems dealing with soil as an engineering material.

SOIL PHYSICS:

The organized body of knowledge concerned with the physical characteristics of soil and with the methods employed in their determinations.

SOIL PROFILE (PROFILE):

Vertical section of a soil, showing the nature and sequence of the various layers, as developed by deposition or weathering, or both.

SOIL STABILIZATION:

Chemical or mechanical treatment designed to increase or maintain the stability of a mass of soil or otherwise to improve its engineering properties.

SOIL STRUCTURE:

The arrangement and state of aggregation of soil particles in a soil mass.

FLOCCULENT STRUCTURE:

An arrangement composed of flocs of soil particles instead of individual soil particles.

HONEYCOMB STRUCTURE:

An arrangement of soil particles having a comparatively loose, stable structure resembling a honeycomb.

SINGLE-GRAINED STRUCTURE:

An arrangement composed of individual soil particles; characteristic structure of coarse-grained soils.

SOIL SUSPENSION:

Highly diffused mixture of soil and water.

SOIL TEXTURE:

See Gradation.

SPECIFIC GRAVITY:

SPECIFIC GRAVITY OF SOLIDS:

G_s, S_s D

Ratio of (1) the weight in air of a given volume of soil solids at a stated temperature

to (2) the weight in air of an equal volume of distilled water at a stated temperature.

APPARENT SPECIFIC GRAVITY:

G_a, S_a D

Ratio of (1) the weight in air of a given volume of the impermeable portion of a permeable material (that is the solid matter including its impermeable pores or voids) at a stated temperature to (2) the weight in air of an equal volume of distilled water at a stated temperature.

BULK SPECIFIC GRAVITY (SPECIFIC MASS GRAVITY):

G_m, S_m D

Ratio of (1) the weight in air of a given volume of a permeable material (including both permeable and impermeable voids normal to the material) at a stated temperature to (2) the weight in air of an equal volume of distilled water at a stated temperature.

SPECIFIC SURFACE:

L^{-1}

The surface area per unit of volume of soil particles.

STABILITY FACTOR (STABILITY NUMBER):

N_s D

A pure number used in the analysis of the stability of a soil embankment. Defined by the following equation:

$$N_s = \frac{H_c \cdot \gamma_e}{c}$$

where: H_c = critical height of the sloped bank,
γ_e = the effective unit weight of the soil, and
c = the cohesion of the soil.

NOTE.—Taylor's stability number is the reciprocal of Terzaghi's stability factor.

STICKY LIMIT:

T_w D

The lowest water content at which a soil will stick to a metal blade drawn across the surface of the soil mass.

STONE:

Crushed or naturally angular particles of rock that will pass a 3-inch sieve and be retained on a No. 4 United States standard sieve.

STRAIN:

e D

The change in length per unit of length in a given direction.

STRESS:

σ, p, f FL^{-2}

The force per unit area acting within the soil mass.

EFFECTIVE STRESS (EFFECTIVE PRESSURE) (INTERGRANULAR PRESSURE):

$\bar{\sigma}$, $\bar{f}$ FL^{-2}

The average normal force per unit area transmitted from grain to grain of a soil mass. It is the stress that is effective in mobilizing internal friction.

NEUTRAL STRESS (PORE PRESSURE) (PORE WATER PRESSURE):

u, u_w FL^{-2}

Stress transmitted through the pore water (water filling the voids of the soil).

NORMAL STRESS:

σ, p FL^{-2}

The stress component normal to a given plane.

PRINCIPAL STRESS:

σ_1, σ_2, σ_3 FL^{-2}

Stress acting normal to three mutually perpendicular planes intersecting at a point in a body, on which the shearing stress is zero.

MAJOR PRINCIPAL STRESS:

σ_1 FL^{-2}

The largest (with regard to sign) principal stress.

MINOR PRINCIPAL STRESS:

σ_3 FL^{-2}

The smallest (with regard to sign) principal stress.

INTERMEDIATE PRINCIPAL STRESS:

σ_2 FL^{-2}

The principal stress whose value is neither the largest nor the smallest (with regard to sign) of the three.

SHEAR STRESS (SHEARING STRESS) (TANGENTIAL STRESS):

r, s FL^{-2}

The stress component tangential to a given plane.

TOTAL STRESS:

σ, f FL^{-2}

The total force per unit area acting within a mass of soil. It is the sum of the neutral and effective stresses.

SUBBASE:

A layer used in a pavement system between the subgrade and base course, or between the subgrade and portland-concrete pavement.

SUBGRADE:

The soil prepared and compacted to support a structure or a pavement system.

SUBGRADE SURFACE:

The surface of the earth or rock prepared to support a structure or a pavement system.

SUBSOIL:

(1) Soil below a subgrade or fill.

(2) That part of a soil profile occurring below the A horizon.

TALUS:

Rock fragments mixed with soil at the foot of a natural slope from which they have been separated.

THERMO-OSMOSIS:

The process by which water is caused to flow in small openings of a soil mass due to differences in temperature within the mass.

THIXOTROPY:

The property of a material that enables it to stiffen in a relatively short time on standing, but upon agitation or manipulation to change to a very soft consistency or to a fluid of high viscosity, the process being completely reversible.

TILL:

See Glacial Till.

TIME FACTOR:

T_v, T D

Dimensionless factor, utilized in the theory of consolidation, containing the physical constants of a soil stratum influencing its time-rate of consolidation, expressed as follows:

$$T=\frac{k(1+e)t}{a_v\gamma_w \cdot H^2}=\frac{c_v t}{H^2}$$

where: k = coefficient of permeability (LT^{-1}),
e = void ratio (dimensionless),
t = elapsed time that the stratum has been consolidated (T),
a_v = coefficient of compressibility (L^2F^{-1}),
γ_w = unit weight of water (FL^{-3}),
H = thickness of stratum drained on one side only (if stratum is drained on both sides, its thickness equals $2H$ (L)), and
c_v = coefficient of consolidation (L^2T^{-1}).

TOPSOIL:

Surface soil usually containing organic matter.

TORSIONAL SHEAR TEST:

A shear test in which a relatively thin test specimen of solid circular or annular cross section usually confined between rings, is subjected to an axial load and to shear in torsion. In-place torsion shear tests may be performed by pressing a dentated solid circular or annular plate against the soil and measuring its resistance to rotation under a given axial load.

TOUGHNESS INDEX:

I_T, T_w

The ratio of (1) the plasticity index to (2) the flow index.

TRANSFORMED FLOW NET:

A flow net whose boundaries have been properly modified (transformed) so that a net consisting of curvilinear squares can be constructed to represent flow conditions in an anisotropic porous medium.

TRANSPORTED SOIL:

Soil transported from the place of its origin by wind, water, or ice.

TRIAXIAL SHEAR TEST (TRIAXIAL COMPRESSION TEST):

A test in which a cylindrical specimen of soil encased in an impervious membrane is subjected to a confining pressure and then loaded axially to failure.

TURBULENT FLOW:

That type of flow in which any water particle may move in any direction with respect to any other particle, and in which the head loss is approximately proportional to the second power of the velocity.

ULTIMATE BEARING CAPACITY:

q_o, q_{ult} — FL^{-2}

The average load per unit of area required to produce failure by rupture of a supporting soil mass.

UNCONFINED COMPRESSIVE STRENGTH:

See Compressive Strength.

UNCONSOLIDATED-UNDRAINED TEST (QUICK TEST):

A soil test in which the water content of the test specimen remains practically unchanged during the application of the confining pressure and the additional axial (or shearing) force.

UNDERCONSOLIDATED SOIL DEPOSIT:

A deposit that is not fully consolidated under the existing overburden pressure.

UNDISTURBED SAMPLE:

A soil sample that has been obtained by methods in which every precaution has been taken to minimize disturbance to the sample.

UNIT WEIGHT:

γ — FL^{-3}

Weight per unit volume.

DRY UNIT WEIGHT (UNIT DRY WEIGHT):

γ_d, γ_o — FL^{-3}

The weight of soil solids per unit of total volume of soil mass.

EFFECTIVE UNIT WEIGHT:

γ_e — FL^{-3}

That unit weight of a soil which, when multiplied by the height of the overlying column of soil, yields the effective pressure due to the weight of the overburden.

MAXIMUM UNIT WEIGHT:

γ_{max} — FL^{-3}

The dry unit weight defined by the peak of a compaction curve.

SATURATED UNIT WEIGHT:

γ_G, γ_{sat} — FL^{-3}

The wet unit weight of a soil mass when saturated.

SUBMERGED UNIT WEIGHT (BUOYANT UNIT WEIGHT):

γ_m, γ', γ_{sub} — FL^{-3}

The weight of the solids in air minus the weight of water displaced by the solids per unit of volume of soil mass; the saturated unit weight minus the unit weight of water.

UNIT WEIGHT OF WATER:

γ_w — FL^{-3}

The weight per unit volume of water; nominally equal to 62.4 pounds per cubic foot or 1 gram per cubic centimeter.

WET UNIT WEIGHT (MASS UNIT WEIGHT):

γ_m, γ_{wet} — FL^{-3}

The weight (solids plus water) per unit of total volume of soil mass, irrespective of the degree of saturation.

ZERO AIR VOIDS UNIT WEIGHT:

γ_z — FL^{-3}

The weight of solids per unit volume of a saturated soil mass.

UPLIFT:

Unit: u — FL^{-2}

Total: U — F or FL^{-1}

The upward water pressure on a structure.

VANE SHEAR TEST:

An inplace shear test in which a rod with thin radial vanes at the end is forced into the soil and the resistance to rotation of the rod is determined.

VARVED CLAY:

Alternating thin layers of silt (or fine sand) and clay formed by variations in sedimentation during the various seasons of the year, often exhibiting contrasting colors when partially dried.

VOID:

Space in a soil mass not occupied by solid mineral matter. This space may be occupied by air, water, or other gaseous or liquid material.

VOID RATIO:

e — D

The ratio of (1) the volume of void space to (2) the volume of solid particles in a given soil mass.

CRITICAL VOID RATIO:

e_c D

The void ratio corresponding to the critical density.

VOLUMETRIC SHRINKAGE (VOLUMETRIC CHANGE):

V_s D

The decrease in volume, expressed as a percentage of the soil mass when dried, of a soil mass when the water content is reduced from a given percentage to the shrinkage limit.

WALL FRICTION:

f' FL^{-2}

Frictional resistance mobilized between a wall and the soil in contact with the wall.

WATER CONTENT:

See Moisture Content.

WATER-HOLDING CAPACITY:

D

The smallest value to which the water content of a soil can be reduced by gravity drainage.

ZERO AIR VOIDS CURVE (SATURATION CURVE):

The curve showing the zero air voids unit weight as a function of water content.

ZERO AIR VOIDS DENSITY (ZERO AIR VOIDS UNIT WEIGHT):

See Unit Weight.

Construction of Embankments

DR. J. W. HILF [1]

E-1. ***General.***—The necessity for control of construction of embankments to impound water has been recognized for many years. In 1932, Justin [1] [2] wrote:

"An entirely safe and substantial design may be entirely ruined by careless and shoddy execution, and the failure of the structure may very possibly be the result. Careful attention to the details of construction is, therefore, fully as important as the preliminary investigation and design."

The consequences of ignoring control are exemplified by the large number of earthfill dams built in the United States during the first quarter of this century which did not survive the first filling of the reservoir. Records show that most of these dams were constructed without moistening the soil and without applying special compactive effort.

The rapid increase in knowledge of soil mechanics since the year 1925 has resulted in substantial progress toward understanding the factors involved in transforming loose earth into a structural material. During this same period, however, the development of huge economical earth-moving machines has increased the placing rate of earthfill many times, thereby intensifying the problem of quality control. Future progress in design economy in the field of earthwork depends not only on advances in soil mechanics and foundation engineering, but also to a large extent on the insistence of inspection personnel on good construction practices in accordance with proper specifications and on their ability to understand and conscientiously apply sound control techniques.

Construction control is obtained by inspection, testing, and reports. The inspector of foundations and earthwork is charged with the responsibility of assuring that work assigned to him is completed in compliance with the specifications. To discharge this responsibility efficiently, he should be fully informed of the design and specifications provisions relating to the work. The attributes of fairness, courtesy, and firmness, coupled with initiative and good judgment, are highly desirable in an inspector. The inspector's diary, containing data on conditions and progress of the work and records of conversations and instructions given to the contractor, is a valuable document that should be carefully preserved.

Proper control of earthwork requires the use of laboratory facilities. For small dams these can be of the portable variety or a small field laboratory can be set up in the vicinity of the site. In most cases commercial laboratory facilities can be used. The control procedures recommended in this text will minimize the cost of control testing consistent with assuring a satisfactory job.

Discoveries of remnants of earthfill dams indicate that man's first engineering structures probably were made of earth. The ancient earthfill dams were constructed by armies of workmen carrying baskets loaded with soil. Excavation was done manually, and some incidental compaction was obtained on the fill by the tramping feet of the porters. Available records do not indicate that there was any intentional moistening or compacting of soil prior to the 19th century. The importance of compaction of earthfill was first evident in England where, by the year 1820, cattle and sheep were used for this purpose. By the middle of the 19th century, heavy smooth rollers made of

[1] Engineer, Earth Dams Section, Bureau of Reclamation.

[2] Numbers in brackets refer to items in the bibliography, sec. E-11.

concrete or metal were in use in Europe and had also been introduced in the United States.

The first sheepsfoot roller, the "Petrolithic" roller, was patented in the United States in 1906 for use in compacting oil-treated road surfacing. The most notable early use of the sheepsfoot roller for compaction of fills started in 1912 in the construction of storage reservoirs by the oil companies of southern California. This type of roller was found to be the only one which compacted the fill in layers and gave uniform compaction without producing laminations. Largely because of the development of the automobile and the airplane, which require roadbeds and airport subgrades of great strength, larger and heavier rollers were developed by the construction industry in the first half of the 20th century.

Published material on moisture control for rolled fills dates back to 1907 when Bassell [2], wrote:

"Too much or too little (water) is equally bad and is to be avoided. It is believed that only by experience is it possible to determine just the proper quantity of water to use with different classes of materials and their varying conditions. In rolling and consolidating of the bank, all portions that have a tendency to quake must be removed at once * * *,"

It was not until 1933 that a definite procedure for moisture and compaction control was established. In a series of articles published in 1933, Proctor [3] gave the principles of soil compaction and their application. Figure 90 shows the Proctor compaction curve, which indicates that for a given compactive effort there is one water content, called the optimum water content, which produces the maximum density or smallest amount of total voids for a given cohesive soil. For greater compactive efforts on the same soil, different moisture-density curves are obtained, whose optimum points occur at smaller water contents and at greater densities than for lesser compactive efforts.

Figure E–1 shows embankment placing operations at Shadow Mountain Dam. The maximum section of this dam is shown in figure 150, and photographs of the completed structure are shown in figure 94 and in the frontispiece.

Figure E–1. **Embankment placing operations in impervious, sand-gravel, and cobble-rockfill zones. The structure is Shadow Mountain Dam on the Colorado River in Colorado. SM–156–CBT.**

E-2. ***Soil Mechanics of Control.*** Compaction of cohesive soils has been definitely proved to follow the principles stated by Proctor. Although there are many kinds of compactive effort used as compaction standards and for compacting cohesive soils, the effect of variation of water content on the resulting unit weight is similar for all methods. Each compactive effort has its own optimum water content. The laboratory standard of compaction used by the Bureau of Reclamation, which has the same intensity of effort as ASTM Designation D 698-42T (see sec. 115(e)), has been found to approximate the actual compaction achieved in the field by 12 passes of the 20-ton dual-drum tamping roller specified in section G-20 (appendix G) on 8- to 9-inch loose lifts (6-inch compacted lifts). The relation between the moisture-density curve for this roller effort on the fill and the standard laboratory compaction curve varies for different soils, but it is close enough so that the laboratory standard can be used for control purposes. Figure E-2 [4] shows the average roller curves for three very different soils used in Bureau dams with their respective laboratory curves.

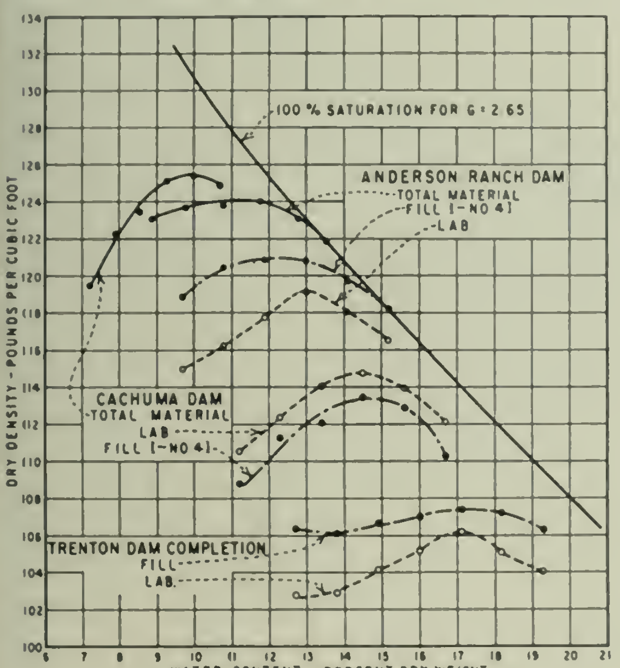


Figure E-2. **Average field and laboratory compaction curves for three dam embankment soils.**

In compacted cohesive soils, permeability, shearing strength, and compressibility are of major concern. It has been shown, both theoretically and by experiment, that an increase in dry unit weight reduces the permeability of a given soil because of the corresponding reduction in amount of voids in the soil mass. From the standpoint of imperviousness, it is desirable to obtain maximum practicable compaction. Extreme impermeability, however, is not always required in the design and, especially in the case of clays, only moderate compactive effort is needed to assure impermeability. On the other hand, well-graded sands and gravels, or even formation rock, can be made quite impervious by the crushing and compacting effort of heavy tamping rollers.

The embankment designs given in chapter V are based on values of angles of internal friction and cohesion obtained by laboratory tests on typical soils. Compaction control has the objective of securing a dry density of soil in the fill sufficient to obtain strength values comparable to those used in these designs. Although the effect of density on cohesion is small in comparison with the effect of variation of water content, test data indicate that the true angle of internal friction of a given soil varies with the compactness of the soil. The angle of internal friction varies among soils because of differences in mineral composition and in the size, shape, and gradation of the soil grains.

For cohesive soils the maximum strength obtainable with a given compactive effort does not occur at that water content which results in the maximum density, but rather at a slightly reduced water content with a density somewhat less than maximum. This condition results because pore pressures produced by compressive forces act to reduce the apparent strength of the soil. These pore pressures increase rapidly with increases in water content in the vicinity of the peak of the compaction curve. Compaction of the soil at water contents slightly less than optimum results in a net increase in strength, because the slight reduction in shearing strength (which accompanies the reduction in density) is more than compensated by the comparatively large reduction in the destabilizing pore pressures which is thereby obtained.

The compressibility of a soil is the relation between effective stress on the soil skeleton and the volume change. Impermeable-type soils vary in compressibility, depending on the amount and character of the fines and according to the amount and gradation of coarse particles they contain. For a particular soil at a given water content, the greater the density the less compressible it will be. The relation between compressibility and development of construction pore-water pressure is such that, for a particular air and water content, the pore pressure increases rapidly with increase in compressibility. In general, a very compressible cohesive soil will develop high pore pressures when loaded unless there is an appreciable amount of air in the compacted soil. The most efficient means of obtaining air in the soil and still have a fairly high density is to compact the soil at a water content slightly less than Proctor optimum.

Coarse-grained, permeable-type soils, also known as cohesionless or free-draining soils, are commonly used as major zones in earthfill dams and as backfill around conduits or behind retaining walls. This type of soil is also used as filter material for drainage in wells and around hydraulic structures. These soils are inherently permeable and have fairly high shearing strengths when compacted. However, in the uncompacted state they are compressible and may be subject to liquefaction upon saturation. The desirable properties of high strength and low compressibility can be greatly improved by compaction of permeable-type soils. Although permeability is thereby decreased, the reduction is usually allowable from a design standpoint.

The most efficient method of compacting cohesionless soils is by vibration when the material is either perfectly dry or when it is nearly saturated with water. The latter method is usually the only practicable one in the field, since perfectly dry material seldom occurs naturally. The shearing strength of permeable materials, such as fairly clean sands and gravels or rockfills, depends almost entirely on the angle of internal friction. Cohesion is negligible, and pore-water pressures are never greater than hydrostatic pressure because of free drainage of the soil. The angle of internal friction is a function of the size, shape, and gradation of the grains, but for a given cohesionless soil its magnitude varies significantly with the void ratio. The state of compactness of the soils is given by their relative density, which is defined in section 115(f).

E-3. ***Preparation of Foundations.***—Foundation design features have been discussed in chapter V, part C. The weak points in earthfill dam construction are generally within the foundation and at the contact of the natural ground surface with placed embankment. Construction of the foundation seepage control and stability features included in the design must be carefully supervised by the inspection force in accordance with the specifications. Dewatering methods used in connection with excavating cutoff trenches or stabilizing the foundations should be carefully checked to see that fine material is not being washed out of the overburden because of improper screening of wells. Whenever possible, well points and sumps should be located outside the area to be excavated to avoid creation of a "live" bottom due to upward flow of water. Concrete footings for cutoff walls or concrete grout caps should be founded on unfractured rock. Blasting for the excavation of these structures should be prohibited or strictly controlled in accordance with the specifications to avoid shattering the foundation.

When overburden is stripped to rock foundations, the rock surface including all pockets or depressions should be carefully cleaned of soil or rock fragments before placing the embankment upon it. This may require handwork and compressed-air cleaning. Rock surfaces which disintegrate rapidly on exposure should be covered immediately with embankment material.

When the foundation is earth, all organic or other unsuitable materials, such as stumps, brush, sod, and large roots, should be stripped and wasted. Stripping operations should be carefully performed to assure removal of all material that may be rendered unstable by saturation, of all material that may interfere with the creation of a proper bond between the foundation and the embankment, and of all pockets of soils significantly more compressible than the average foundation material. Stripping of pervious materials under the pervious or semipervious zones of an embankment should be limited to the removal of surface debris and grass roots. Test pits for further exploration should be excavated if the stripping operations indicate the presence of unstable or otherwise unsuitable material, and an inspection should be made by an experienced engineer.

Prior to placing the first layer of embankment on an earth foundation, moistening and compacting the surface by rolling with a tamping roller is necessary to obtain proper bond. Rock foundation surfaces should be moistened, but no standing water should be permitted when the first lift is placed. Sometimes the foundation surface requires scarification by harrows to assure proper bonding; however, no additional scarification is usually necessary if it is penetrated by tamping rollers. Where the rock foundation would be injured by penetration of the tamping roller feet, it is permissible to use a thicker-than-specified layer of earthfill for the first compacted layer. However, the first layer should never exceed 15 inches loose lift for 9-inch-long tamper feet, and additional roller passes are required to insure that proper compaction is obtained. Special compaction methods such as hand tamping should be used in pockets that cannot be compacted by the specified roller, instead of permitting an unusually thick initial lift to obtain a uniform surface for compaction.

Figure E–3 shows power tamping operations along the contact of the earthfill portion of a dam with the rock abutment. The irregular surface of the rock prevents proper compaction by means of rollers.

The use of very wet soil for the first lift against the foundation should generally be avoided; rather, the foundation should be properly moistened. On steep, irregular rock abutments material wetter than optimum may be necessary or desirable in order to obtain bond. However, the use of such material should be permitted only in special cases with the approval of the contracting authority. Care must be exercised when special compaction is used to insure that bond is created between successive layers of material. This may require light scarification between lifts of tamped

Figure E–3. **Power tamping of earthfill at contact with irregular surface of rock abutment. This structure is Green Mountain Dam on the Blue River in Colorado. GM–375–CBT.**

material. Appendix G contains sample specifications pertinent to items of work required for preparation of foundations.

E-4. *Earthfill.*—Specifications provisions for control of placement, water content, and compaction of earthfill are given in appendix G. These must be implemented by establishing procedures to insure their attainment. For the construction of small dams within the scope of this text, the plan of control for embankments of cohesive soil is to place the material at the Proctor optimum water content and maximum laboratory density. Optimum water content rather than a water content slightly less than optimum is selected for the reasons given in section 131. The most important variables affecting construction of earthfill embankments are distribution of soils, placement, water content and its uniformity throughout the spread material, water content of the borrow material, methods for correcting borrow material water content if too wet or too dry, roller characteristics, number of roller passes, thickness of layers, maximum size and quantity of gravel sizes in the material, condition of the surface of layers after rolling, and effectiveness of power tamping in places inaccessible or undesirable for roller operation.

Figure E-4 shows placing, leveling, and compacting of the semipervious zone of the embankment at Olympus Dam. Compacting was done by tamping rollers, as the material was not pervious enough to permit compaction as a pervious fill in the manner described in section E-6. The maximum section of this dam is shown on figure 145, and a photograph of the completed structure is shown as figure 22.

Adequate inspection and laboratory testing are essential to maintain the control of earthfill construction. Attention is called to the fact that it is impossible even for an experienced soils engineer to determine visually the degree of compaction of cohesive soil, especially when it is dry of optimum. The apparent cohesion of these soils makes them firm and gives them the appearance of

Figure E-4. **Placing, leveling and compacting the fill on Olympus Dam. A combination earthfill and concrete gravity dam on the Big Thompson River in Colorado. 375-EPA-PS.**

denseness which disappears when the soil becomes saturated. There is no satisfactory substitute for control testing to determine the degree of compactness for these soils. The testing must include all critical areas where seepage or loss of strength may induce failure.

Borrow pit inspection includes controlling and recording all earthwork operations prior to dumping the material on the embankment. Areas to be excavated are selected, depths of cut are determined, and the zone of the dam in which a particular material should be placed is predetermined. The borrow pit inspector checks the adequacy of any mixing or separation methods used by the contractor. As required, he cooperates with the contractor in determining the amount of water to be added to the borrow pit by irrigation or to be removed by drainage to attain proper water content of the materials prior to placing. Either the rapid method of compaction control given in this appendix or the Proctor needle value (fig. 90) obtained in a cylinder of compacted minus No. 4 fraction of soil, can be used to determine the status of natural moisture conditions in the borrow pit with respect to the optimum water content. Every effort should be made to have the excavated material as close as possible to the optimum water content prior to delivery on the embankment.

The embankment inspector should be provided with a means for determining the location and elevation of tests made on the embankment and for reporting the location of the contractor's operations. Horizontal control by means of coordinates or stations and offsets should be established. It has been found desirable to establish vertical control by benchmarks and by the use of stadia rods from which the inspector can establish an elevation anywhere on the fill by using a hand level. When materials are brought on the embankment, the inspector should see that they are placed in the proper zones. If a zoned embankment is being constructed, lines of demarcation can be painted on rock abutments or marked by flags. Within a particular zone or on homogeneous embankments, the objective is to direct the placing of materials so that the most impervious soils are located in the center of the impervious zone and the coarser, more pervious soils are placed toward the slopes of the embankment, so that the permeability and stability of materials will increase toward the outer slopes. In general, when materials differ in dry density but have about the same percolation rate, the material having the greater dry density should be placed in the outer sections of the zone or of the dam as the case may be.

After the materials are placed in the proper location, the embankment inspector determines whether they contain the proper amount of moisture prior to compaction. *This is of utmost importance.* The rapid compaction control method or the Proctor needle value should be used for this determination. Should the materials arrive on the embankment too dry, it will be necessary to condition them by sprinkling prior to, during, or after spreading. The contractor's operations in sprinkling and mixing the moisture with the soil will vary, but it is of paramount importance that the proper water content be uniformly distributed throughout the spread layer prior to rolling.

Another important inspection task is the determination of the thickness of the compacted layer. A layer that is spread too thick will not give the desired density for given compaction conditions. Initial placing operations will determine the proper spread thickness of a layer that will compact to the specified thickness. This is usually 8 to 9 inches for a 6-inch compacted lift of earthfill. A method of determining average thickness of placed layers is to take and plot daily a cross section of the fill at a reference station. The inspector's report for that day should contain the number of layers placed at that section, from which the average thickness can be determined.

The removal of oversized rock from earthfill embankment material when the oversized rock content is greater than about 1 percent is most efficiently done prior to delivery of the soil on the embankment. This procedure was used at Crescent Lake Dam, shown in figure E-5. The shovel at the left in the photograph is excavating and mixing the borrow material. The scoopmobile transports the material to electrically operated screens, which separate the oversize rock from the soil; trucks are loaded by means of a conveyor belt.

Smaller amounts of oversized rock can be removed by hand picking or, under favorable conditions, by various kinds of rock rakes. Oversized rock that has been overlooked prior to rolling can generally be detected by the inspector during roll-

Figure E-5. **Operations in impervious borrow area for Crescent Lake, a small earthfill storage dam on Crescent Creek in Oregon. Oversize rock is being removed by screening pit-run material. P806-126-55.**

ing, by his observing the bounce which occurs when the roller passes over the hidden rock. The inspector should assure that the rock is removed from the fill.

The inspector is charged with the responsibility of assuring that the specified number of roller passes is made on each lift. An oversight in maintaining the proper number of passes may lead to considerable drop in the desired degree of compaction. The insistence of orderliness on the fill and the establishment of routine construction operations will minimize trouble from too few roller passes.

The final check on the degree of compaction attained is done by the rapid method of compaction control given in section E-5. If the field dry density of the material passing the No. 4 screen is above the minimum allowable density as given in section E-10, and if the water content is within the allowable limits, the embankment will be ready for the next layer after such scarifying and moistening as may be necessary to secure a good bond between the layers.

Mechanical tamping, when used around structures, along abutments, and in areas inaccessible to the rolling equipment, should be carefully watched and checked by frequent density tests. The procedure to be followed for mechanical tamping will depend greatly on the type of tamper used. Some of the factors affecting density are thickness of the layer being placed, time of tamping, air pressure if air tampers are used, water content of the material, and weight of the tamping unit.

An important function of inspection is to determine when and where to make field density tests. These tests should be made (1) in areas where the degree of compaction is doubtful, (2) in areas where embankment operations are concentrated, and (3) for every 2,000 cubic yards of embankment when no doubtful or concentrated areas occur. Areas susceptible to insufficient compac-

tion include the junction between mechanical tamping and rolled embankment, along abutments or cutoff walls; areas where rollers turn during rolling operations; areas where too thick a layer is being compacted; areas where improper water content exists in a material; and areas where less than the specified number of roller passes are made.

When embankment operations are concentrated in a small area (that is, if many layers of material are being placed one over the other in a single day), tests should be made in this area in every third or fourth layer to assure that the desired density is being attained. If areas of doubtful compaction do not exist and no tests are required because of concentrated areas, at least one field density test should be made for each 2,000 cubic yards of compacted embankment and it should be representative of the degree of compaction being obtained.

E–5. *Rapid Compaction Control Method.*—The density-in-place test described in section 114 determines the dry density and the water content of the compacted fill. For control purposes these values must be compared with the laboratory maximum dry density and the optimum water content. The rapid control procedure described herein yields the exact percentage of laboratory maximum dry density and a very close approximation of the difference between fill water content and optimum water content of a field density sample, without requiring knowledge of water contents. The theoretical basis for this method was described by Hilf [5]. With this method, compaction control can be effected within 1 hour from the time the field test is made. For record purposes one water content (the fill water content) is measured, and after it is known (usually the following day), the values of fill dry density, cylinder dry density at fill water content, laboratory maximum dry density, and optimum water content are computed.

The equipment required for the rapid compaction control method, using mechanical mixing and drying, is listed below and is shown on figure E–6. Each item is listed according to its number on the figure. The asterisks indicate mechanical equipment not required when hand mixing methods are used. Items 3 and 4 are needed only when it is desired to control water content with the Proctor needle rather than the rapid method procedure.

1. Scale, 30-pound capacity, graduated in 0.01 pound.
2. Scale weights.
3. Penetration resistance needles.
4. Penetration resistance gage (penetrometer).
5. Wood mallet.
6. 5.5-pound compaction hammer with 18-inch drop guide.
7. Large butcherknife.
8. Wire brush.
9. Small hand scoop.
10. Mixing pan.
11. Clipboard for data sheet.
*12. Electric fan.
13. Hand scoop.
14. Glass graduate, 100-ml. capacity.
*15. Mixing paddle, type III.
*16. Mixer bowl with collar, 3-gallon capacity.
*17. Mixer (with ⅓-horsepower, 60-cycle, 115-volt motor).
18. Towel.
*19. Mixing paddle type II.
20. Straightedge trimmer.
21. Spanner wrench for compaction cylinder.
22. Base for compaction cylinder, concrete-filled carbide can (or equivalent).
23. $\frac{1}{20}$-cubic-foot compaction cylinder with base plate and collar.

The form suggested to obtain rapid control values is shown in figure E–7. A 1-minute mixing period with mechanical equipment using a paddle speed of 90 revolutions per minute has provided uniform distribution of water through the 7.5-pound sample. A type III paddle shown in figure E–6 may be used for all soils; a type II paddle provides satisfactory mixing of low-plasticity silts. Water is most efficiently added to the sample by pouring it on the surface before starting the mixer. Hand mixing in the pans should be done as rapidly and thoroughly as practicable. Drying a soil sample when necessary to remove water may be accomplished rapidly by blowing air across the bowl while slowly mixing the sample. The standard compaction method given in section 115(e) is used.

It should be noted that the form shown in figure E–7 is designed for the Bureau of Reclamation's laboratory compaction standard using the $\frac{1}{20}$-

Figure E–6. **Equipment used for rapid compaction control method.**

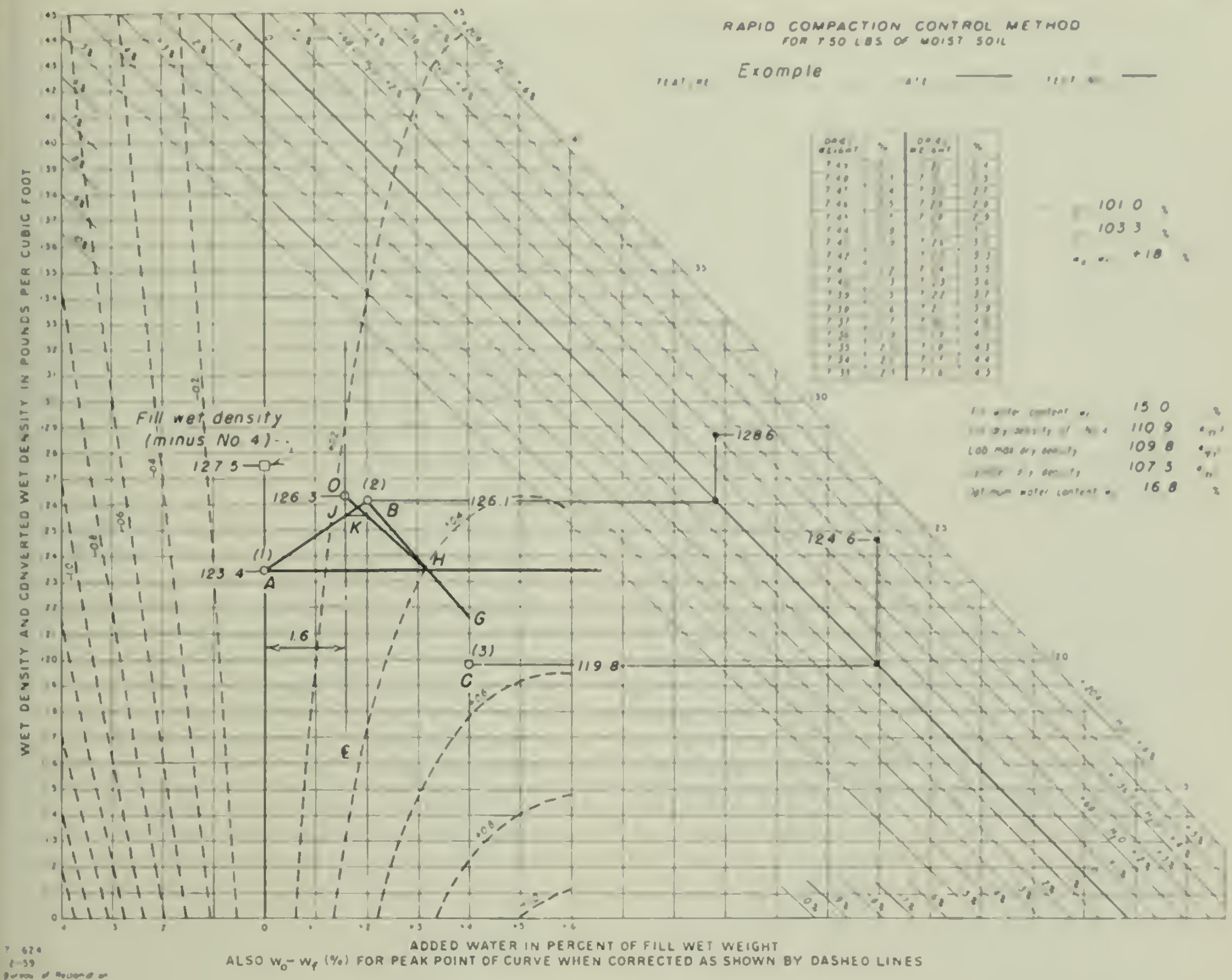


Figure E–7. **Form used for rapid compaction control method.**

cubic-foot cylinder. By changing the weights of soil sample and of water added in proportion to the size of cylinder used, this form is suitable for density control for any compaction standard. Also, for most soils the correction curves for moisture control can be used for other compaction standards of approximately the same compactive effort, such as the ASTM Designation D 698–42T and the standard AASHO T99–49 method. For moisture control of soils with very low specific gravities (lower than 2.50) the curves may have to be modified or the optimum water content may have to be estimated using the equations of the rapid compaction control method [5].

The rapid compaction control method involves the determination of the following:

$$D=\frac{\text{Fill dry density}}{\text{Laboratory maximum dry density}}$$

$$C=\frac{\text{Fill dry density}}{\text{Cylinder dry density at fill water content}}$$

$w_o - w_f$ (optimum water content minus field water content) in percent

Ratios D and C are actually obtained on the basis of corresponding wet densities. As a first step the fill wet density of the minus No. 4 fraction is obtained (see fig. 88). Then three or four points are computed and plotted on the moisture versus "converted wet density" curve as follows:

(1) To obtain point 1: Compact soil at fill water content into a standard cylinder. Plot

the resulting wet density on the 0 percent vertical line on the form shown in figure E–7.

(2) To obtain point 2: To 7.50 pounds of soil at fill water content add 68 cc. (2 percent) water, and mix and compact into a cylinder to determine corresponding wet density. Find the point on the +2 percent diagonal line corresponding to this wet density; project vertically to the 0 percent diagonal line, thence horizontally to plot point 2 on the +2 percent vertical line. The ordinate of the plotted point is the wet density divided by 1.02.

(3) To obtain point 3 if point 2 is greater in ordinate than point 1: To 7.50 pounds of soil at fill water content add 136 cc. (4 percent) water, mix and compact into a cylinder. Find the point on the +4 percent diagonal line corresponding to the wet density; project vertically to the 0 percent diagonal line, thence horizontally to plot point 3 on the +4 percent vertical line. The ordinate of the plotted point is the wet density divided by 1.04.

(4) To obtain point 3 if point 2 is smaller in ordinate than point 1: Permit 7.50 pounds of soil at fill water content to dry without loss of soil; then weigh.[3] The table on the right-hand portion of the form gives the percentage of water loss corresponding to the dried weight. Compact the dried soil into a cylinder. Find the point on the diagonal line (interpolate if necessary) corresponding to the wet density; project vertically to the 0 percent diagonal line, thence horizontally to plot point 3 on the vertical line corresponding to the correct percentage. The ordinate of the plotted point is the wet density divided by 1 plus the negative percentage (e.g. $1+(-0.02)=0.98$).

(5) Three plotted points are sufficient if both the left and right points are lower in ordinate than the center point; if not, a fourth point is necessary. Find point of maximum ordinate of the curve by the parabola method shown in figure E–8, or by sketching the curve if the number and locations of the points permit accuracy without use of the parabola method.

(6) Plot the fill wet density of minus No. 4 fraction on the 0 percent vertical line.

(7) *To obtain D:* Project the maximum ordinate (obtained in step 5) horizontally to the 0 percent diagonal line, thence vertically to the value of the fill wet density. $D=100$ percent plus the interpolated percentage given by the diagonal lines, taking minus signs into account. (D is the fill wet density divided by the maximum ordinate of the curve.)

(8) *To obtain C:* Project point 1 horizontally to the 0 percent diagonal line, thence vertically to the value of the fill wet density. $C=100$ percent plus the interpolated percentage given by the diagonal lines, taking minus signs into account. (C is the fill wet density divided by the ordinate of point 1.)

(9) *To obtain* w_o-w_f: This value is the abscissa of the point of maximum ordinate corrected by adding the value shown in red (the dashed lines) on the chart nearest to the peak point, interpolating where necessary, and taking minus signs into account. Abscissa and correction values should be taken to the nearest 0.1 percent.

(10) Rapid control values obtained in steps (7), (8), and (9) yield sufficient information to accept or reject the work.

For record purposes, dry a sample of minus No. 4 fraction in an oven at 110° C. to obtain actual fill water content, w_f. Then:

Fill dry density of minus No. 4 = (Fill wet density of minus No. 4) $\div (1+w_f)$

Laboratory maximum dry density = (Maximum ordinate) $\div (1+w_f)$

Cylinder dry density = (Ordinate of point 1) $\div (1+w_f)$

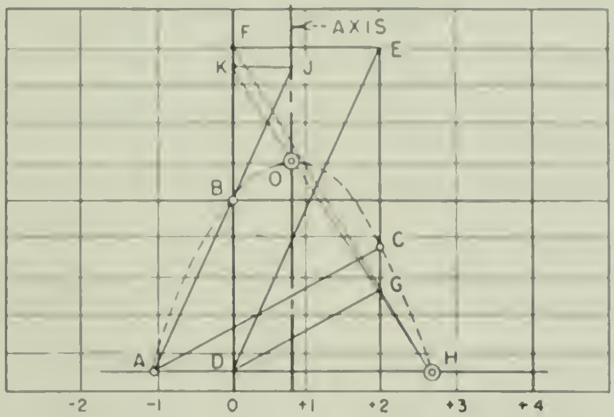
Optimum water content $= w_f + (1+w_f) \times$ (abscissa at maximum ordinate).

Two examples will illustrate the method: In example 1, the fill water content is less than optimum. The test data are shown in figure E-7.

[3] If point 2 is within 3 pounds per cubic foot of point 1, the requirement for drying may be eliminated by using the alternative procedure given at the end of this section.

Parabola Method

Graphical solution for vertex, O, of a parabola whose axis is vertical, given three points A, B, and C. If more than three points are available, use the three closest to optimum.



1. Draw horizontal base line through the left point, A, and draw vertical lines through points B and C.
2. Draw line DE parallel to AB, point E lies on the vertical line through point C, project E horizontally to establish point F on the vertical line through B.
3. Draw line DG parallel to AC, point G lies on the vertical line through point C.
4. Line FG intersects the base line at H. Axis of parabola bisects AH; draw the axis.
5. Intersection of line AB with the axis is at J; project J horizontally to K, which lies on the vertical line through point B.
6. Line KH intersects the axis at O, the vertex.

NOTE: If points A, B, and C are equally spaced horizontally (this is true when 2 points are obtained by adding water or when soil is dried exactly 2 percent) steps 2 and 3 above are eliminated. Point F coincides with point B and point G is halfway between the base line and point C. Hence, point H is obtained by drawing BG and point O is obtained by steps 5 and 6 as usual. See graph below.

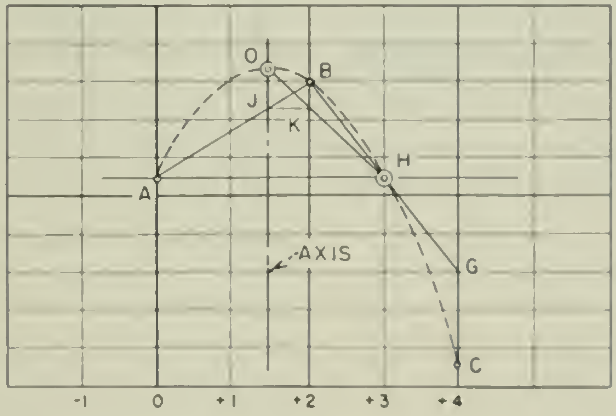


Figure E-8. Graphical method of finding peak point of converted wet density curve.

Fill wet density (minus No. 4 basis) = 127.5 pounds per cubic foot.

Point	Wet density, pounds per cubic foot	Abscissa, percent	Converted wet density, pounds per cubic foot
1	123.4	0	123.4
2	128.6	2	126.1
3	124.6	4	119.8
By parabola method:			
0		1.6	126.3

Then:

$$D=\frac{127.5}{126.3}=101.0 \text{ percent}$$

$$C=\frac{127.5}{123.4}=103.3 \text{ percent}$$

$$w_o-w_f=+1.6+0.2=+1.8 \text{ percent (dry).}$$

After the fill water content has been determined by drying a sample to constant weight at 110° C., the field density test is completed for record purposes as follows:

$w_f=15.0$ percent (determined by drying at 110° C.)

$$\gamma_{D_f}=\frac{127.5}{1.15}=110.9 \text{ lb. per cu. ft.}$$

$$\gamma_{D_m}=\frac{126.3}{1.15}=109.8 \text{ lb. per cu. ft.}$$

$$\gamma_{D_c}=\frac{123.4}{1.15}=107.3 \text{ lb. per cu. ft.}$$

$$w_o=0.15+(1.15)(0.016)=16.8 \text{ percent.}$$

In example 2, the fill water content is greater than optimum. The test data are as follows:

Fill wet density = 125.8 pounds per cubic foot.

Point	Wet density, pounds per cubic foot	Abscissa, percent	Converted wet density, pounds per cubic foot
1	128.4	0	128.4
2	124.2	+2	121.8
3	123.7	−2.3	126.6
By parabola method:			
0		−0.7	128.9

Then:

$$D=\frac{125.8}{128.9}=97.6 \text{ percent}$$

$$C=\frac{125.8}{128.4}=98.0 \text{ percent}$$

$$w_o-w_f=-0.7-0.1=-0.8 \text{ percent (wet).}$$

Completion of test for record purposes:

$w_f=18.0$ percent

$$\gamma_{D_f}=\frac{125.8}{1.18}=106.6 \text{ lb. per cu. ft.}$$

$$\gamma_{D_m}=\frac{128.9}{1.18}=109.2 \text{ lb. per cu. ft.}$$

$$\gamma_{D_c}=\frac{128.4}{1.18}=108.8 \text{ lb. per cu. ft.}$$

$$w_o=0.18+(1.18)(-0.007)=17.2 \text{ percent.}$$

Penetration resistance needle readings required for purposes of maintaining moisture control by means of the needle-moisture relation shown on figure 90 may be obtained during the rapid method test. The needle readings in the compaction cylinder at fill water content may be obtained directly after the weighing required to determine point 1 of the rapid method. To obtain the Proctor needle value at optimum water content, needle readings corresponding to points 2, 3, etc., should be obtained after weighing. All needle readings may then be plotted on figure E–7 to any convenient ordinate scale. The needle value at optimum is the intersection of a vertical line through the peak point of the converted wet density curve and the Proctor needle curve.

The following alternative method may be used to eliminate the drying requirement for soils that are close to optimum in water content.

Points 1 and 2 of the converted wet density curve are obtained in the usual manner. To obtain point 3 when point 2 is smaller in ordinate than point 1 but within 3 pounds per cubic foot of that point: In lieu of drying 7.50 pounds of soil at fill water content as required in the procedure for the rapid compaction control method, add 34 cubic centimeters (1 percent) of water to this quantity of soil; mix, and compact into a cylinder. Find the point on the +1 percent diagonal line corresponding to the wet density, project vertically to the 0 percent diagonal line, thence horizontally to plot point 3 on the +1 percent vertical line (fig. E–7). The ordinate of the plotted point is the wet density divided by 1.01.

If point 3 thus obtained is greater in ordinate than point 1, the peak point of the converted wet density curve can be obtained graphically by the parabola method. If point 3 is smaller in ordinate than point 1, the graphical procedure for obtain-

ing the peak point requires extrapolation which reduces its accuracy. For this case, calculate Y_1/Y_2, where Y_1 is the difference in ordinates of points 1 and 3, and Y_2 is the difference in ordinates of points 1 and 2. Figure E–9 gives the location of the axis of the parabola (distance z_m from the origin) for ratios Y_1/Y_2.

For example, if points 1, 2, and 3 are equal to 117.5, 115.0, and 116.6 pounds per cubic foot, respectively,

$Y_1 = 117.5 - 116.6 = 0.9$,

$Y_2 = 117.5 - 115.0 = 2.5$, and

$$\frac{Y_1}{Y_2} = \frac{0.9}{2.5} = 0.36.$$

From figure E–9, $z_m = -0.8$ percent.

With the axis of the parabola determined, the mirror images of points 1, 2, and 3 can be plotted as shown by points 1′, 2′, and 3′ in figure E–10. The peak point of the parabola, point 0, is obtained by designating point 3′ as A, point 1′ as B, and point 3 as H and proceeding with the parabola construction. The coordinates of the peak point of the converted wet density curve are (−0.8, 117.7).

E–6. *Pervious Fill.* Permeable-type materials are used in rolled earthfill dams to provide an outer shell of high strength to support the impervious core, to secure favorable hydraulic conditions of drainage, and to act as filters between materials of wide variations in grain sizes. Control of con-

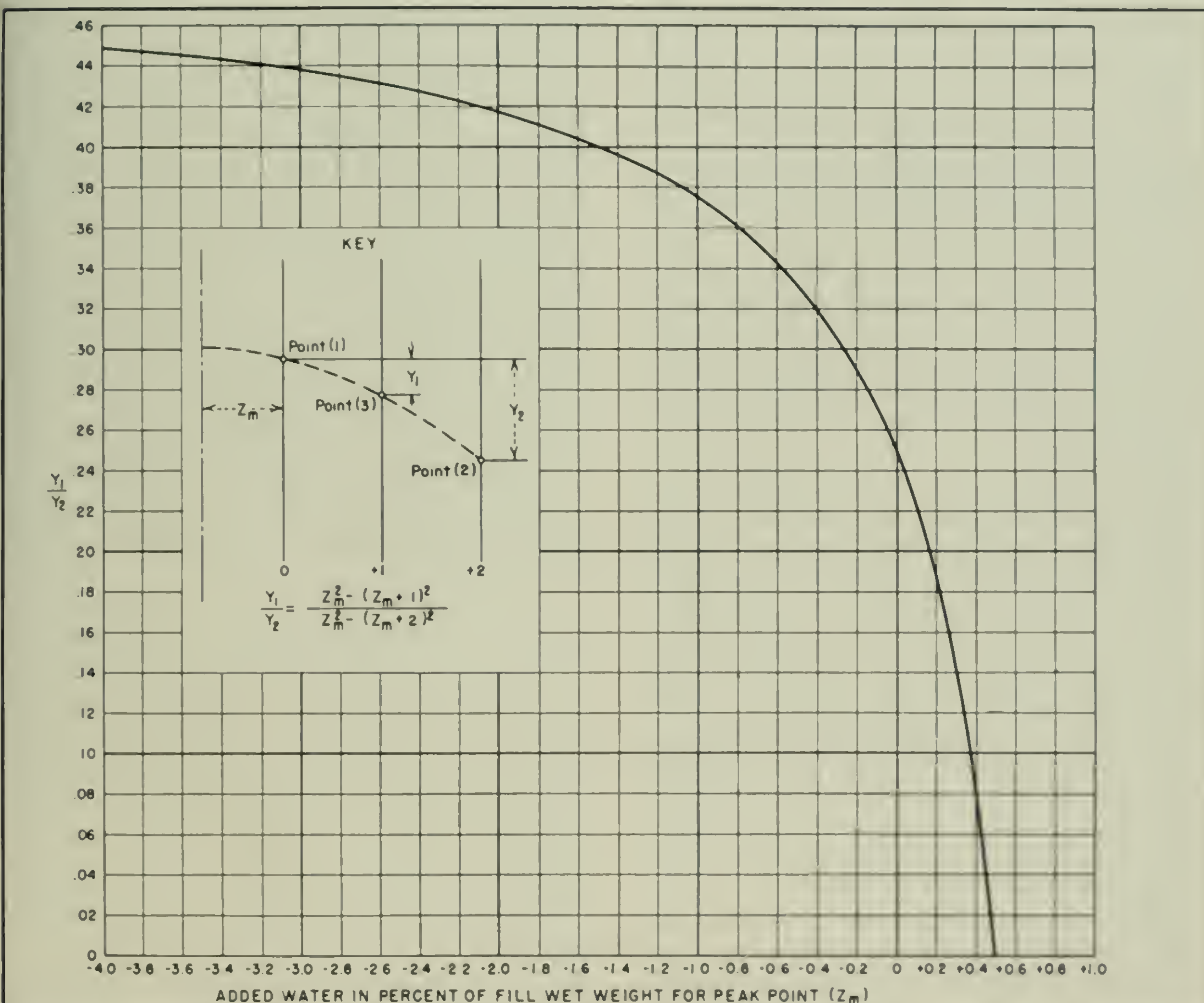


Figure E–9. **Alternative procedure to eliminate requirement for drying soils that are close to optimum water content.**

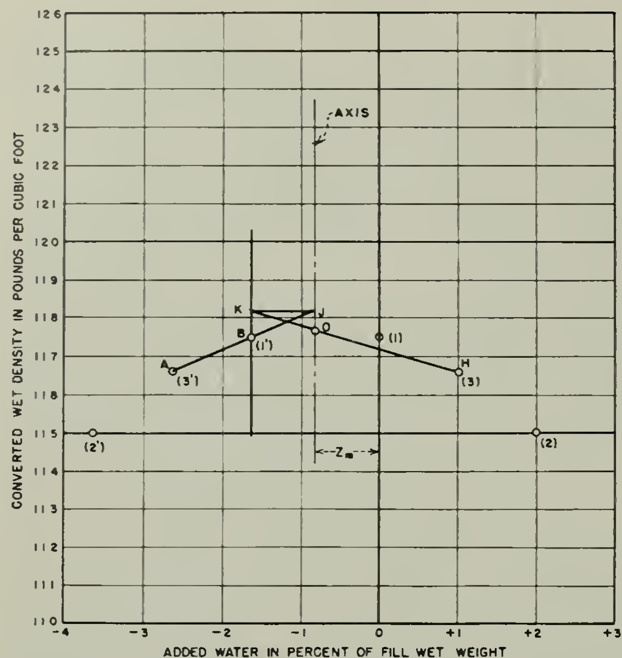


Figure E–10. Example of parabola construction for alternative method which eliminates drying requirement for a soil that is close to optimum water content.

struction of zones of sand and gravel is necessary to assure that (1) the material is formed into a homogeneous mass free from large voids, (2) the soil mass is free draining, (3) the material will not consolidate excessively under the weight of superimposed fill, and (4) the soil has a high angle of internal friction.

The workability of a permeable-type soil is reduced considerably by the inclusion of even small amounts of silt or clay; hence, every effort should be made to insure that the contractor's operations in the borrow pits and on the fill are such that contamination of the pervious soil is held to a minimum. As the material is brought on the fill, it is directed to the proper zone. Within the pervious zone, individual loads should be placed so that the material will be graded in coarseness toward the outer slopes. When compacted thicknesses are specified, the thickness of loose layers should be determined by the inspector during the initial stages of construction. Since the field density will be checked by relatively few actual tests after satisfactory placing procedures have been established, the proper thickness of the loose layers must be maintained within close limits throughout the job. The thickness of specified compacted layers is usually made large enough to accommodate the size of rock encountered in the borrow area. In cases where cobbles or rock fragments greater in size than the specified thickness of layer occur, provisions are usually made for special embedding, removal to outer slopes of the pervious zone, or removal to other zones. In order to secure the best compaction, the inspector should see that the specified provisions for disposal of oversized rock are followed.

After the material has been placed and spread to the desired thickness of lift and oversized cobbles or rock fragments have been disposed of, the next important step is application of water. Thorough and uniform wetting of materials during or immediately prior to compaction is essential for best results. The most applicable method of adding and distributing water on the fill should be determined during the initial placement. It has been found that relaxation of the requirements of thorough wetting may result in densities far below the minimum requirements even with excessive compactive effort. Different pervious materials require different amounts of water for thorough wetting and best compaction. In general, it is desirable to add as much water to the material as it will readily absorb. Too much water cannot be added to an extremely pervious soil; however, permeable-type soils containing small amounts of silt or clay may become temporarily boggy if an excessive amount of water is used. For these soils care must be exercised in wetting the soil. The contractor's operations should be carefully controlled to avoid excessive wetting of the impervious zone adjacent to the pervious material being compacted.

When compacting permeable-type soils by the treads of crawler-type tractors, it is desirable to have the tractor operating at the highest practicable speed during the compaction operation. High speed is conducive to greater vibration which aids in the compaction. In inspection of compaction operations using tractor treads, it is important to require the tractor to make a complete coverage of the area to be compacted prior to making the second and subsequent passes. Different widths of areas to be compacted will require different numbers of tractor trips to obtain the same number of passes of the treads. The proper number of trips should be determined and enforced.

It is recommended that relative density tests and mechanical analyses be made during the initial placing operation at a frequency of about one test representing each 1,000 cubic yards placed. The procedure for making relative density tests is given in sections 114(a) and 115(f). After placement procedures have proved satisfactory, one relative density test for every 10,000 cubic yards of material placed will suffice unless significant changes in gradation occur. In the event of significant gradational changes in the borrow material, increased frequency of field tests may be needed to insure satisfactory compaction of the variable materials.

E-7. *Rockfill and Riprap.* —Rockfill zones are used in earthfill dams to provide stability for the embankment and to protect exposed surfaces of the fill. Riprap is a relatively thin layer of specially selected and graded rock fragments used for protecting earth slopes from erosion by water currents and waves. Rockfill and riprap are usually not compacted but are dumped or placed so as to obtain high shearing strength by interlocking of the angular fragments. The most desirable riprap surface is well keyed but rough in order to resist wave action effectively. High permeability is desirable in rockfills; hence, the amount of fines permitted is limited. On the other hand, large unfilled voids are undesirable. The outer portion of a rockfill zone should contain the largest available rock in order to secure slope protection. Where very large rockfill sections are used, excessive settlement may be a problem, and sluicing may be required to compact the fill.

Inspection may be necessary both at the rock source and on the rockfill to insure that the material is selected to avoid an excessive amount of fines in the rockfill. Breakage in handling and transporting should be taken into account. Placing operations should be inspected to see that segregation is avoided and that no large voids are left in the rockfill. If sluicing is required, the contractor's operations should be carefully controlled to avoid excessive wetting of the impervious zone and to insure that a sufficient quantity of water is being used uniformly. Inspection of riprap placement consists of visual observation of the operation and of the completed product to insure that a dense, rough surface of well-keyed, graded rock fragments of the specified quality and sizes is obtained. Typical specifications provisions for placing rockfill and riprap are contained in appendix G.

E-8. *Miscellaneous Fills.* Dam embankments on saturated fine-grain foundations may require toe support fills, the weight of which acts to improve stability. These are discussed in section 129. Excavation for foundations of the dam or for the appurtenant structures often produces material unsuitable for or in excess of the requirements for the structural zones of the dam. Such excavated material can be used for stabilizing fills at the toes of the dam. In localities where riprap of good quality is very expensive, fill materials from structural excavations have been used to flatten the upstream slope of the dam so as to permit use of poor quality rock or, in some case, the omission of rock. In a few cases excess required excavation has been used in an isolated zone in the downstream portion of a dam merely to replace material which otherwise would have to be borrowed at greater expense.

The permeability of stabilizing fills is not important in the design, and such fills usually are not specially compacted. However, full use should be made of the compaction obtainable by routing of the hauling and placing equipment over layers of the material. Sometimes the nature of the available materials or the design requires that some compactive effort other than routing of equipment be used. For example, sheepsfoot rolling has been used to break up fairly large chunks of soft rock to avoid excessive settlement. Compaction also may be required when the miscellaneous fill is to serve as an impervious blanket.

Inspection of miscellaneous fills is usually entirely visual; ordinarily no control tests are made. The main problem in inspection of miscellaneous fills is to assure that the specified thickness of lift is not exceeded and to see that the hauling equipment is not channelized by a roadway but is spread over the entire placed area so far as practicable.

E-9. *Records and Reports.* Daily reports should be made by the inspector covering the activities for his shift. These reports should record the progress of construction, provide pertinent information for the inspector about to go on shift (including shutdowns and orders given to the contractor), and furnish data for use in compiling reports. The form of a daily report will vary to

suit the requirements of each job, but all information called for on summary progress reports should be based on day-to-day records. It is desirable that a systematic method of identifying field density tests made on the embankment be used. A suggested scheme is to designate each test by the date, shift, number on that shift, and purpose. For example, "8–2–58–a–2–D" would define a field density test made August 2, 1958, on the first shift, the second test made on that shift, and for the purpose of checking an area of doubtful compaction. The legend is as follows: a, first shift; b, second shift; c, third shift; D, doubtful area; C, concentrated area; R, representative. The results of tests made daily on the embankment can be reported on appropriate forms.

E–10. ***Control Criteria.***—Determination of the quality of the embankment being placed can be made by a simple statistical analysis of the test results as given by Davis [6]. Figures E–11 and E–12 show work sheets and curves for dry density control and moisture control, respectively, for compacted cohesive soil in an earthfill dam. By means of this analysis, the frequency distribution of the test results is obtained from which statistical values such as the mean, standard deviation, and the percentage of tests falling outside specified limits can be determined.

Various criteria for quality control have been proposed. Table E–1 lists suggested limits of density and moisture control based on experience gained in compacting 44 cohesive soils and 18 cohesionless soils in Bureau of Reclamation earthfill dams. The soils were compacted by the equipment and methods specified in appendix G; hence, the values given in the table may not be possible of attainment with other methods of compaction or with lesser amounts of compactive effort. It is recognized that the normal frequency distribution curve for any desired average value permits a small percentage of very low tests. However, because of the relatively small number of samples recommended to be tested, the values listed in the table as "minimum acceptable" are suggested as a basis for requiring recompaction of the area represented by lower test values.

The effect of gravel content in cohesive soils is discussed in several papers [4, 7, 8]. Available data indicate that lesser percentages of density on the minus No. 4 basis are required for gravelly cohesive soils than for soils containing little or no gravel. This fact is reflected in table E–1.

Table E–1.—*Criteria for control of compacted dam embankments*

Type of material	Percentage of +No. 4 fraction by weight of total material	Percentage based on minus No. 4 fraction		
		Minimum acceptable density	Desirable average density	Moisture limits, w_o–w_f
Cohesive soils controlled by Proctor test.	0–25	D=95	D=98	−2 to +2.
	26–50	D=92.5	D=95	
	More than 50 [1]	D=90	D=93	
Cohesionless soils controlled by relative density test.	Fine sands with 0–25.	D_d=75	D_d=90	Soils should be very wet.
	Medium sands with 0–25.	D_d=70	D_d=85	
	Coarse sands and gravels with 0–100.	D_d=65	D_d=80	

w_o–w_f is the difference between optimum water content and fill water content in percent of dry weight of soil.

D is fill dry density divided by Proctor maximum dry density, in percent.

D_d is relative density, as defined in appendix D, in percent.

[1] Cohesive soils containing more than 50 percent gravel sizes should be tested for permeability of the total material if used as a water barrier.

E–11. *Bibliography.*

[1] Justin, J. D., "Earth Dam Projects," John Wiley & Sons, Inc., New York, N.Y., 1932, p. 188.

[2] Bassell, Burr, "Earth Dams," Engineering News Publishing Co., New York, N.Y., 1907.

[3] Proctor, R. R., "The Design and Construction of Rolled Earth Dams," Engineering News-Record, Aug. 31, Sept. 7, 21, and 28, 1933.

[4] Hilf, J. W., "Compacting Earth Dams With Heavy Tamping Rollers," ASCE Proceedings, vol. 83, April 1957, Journal of the Soil Mechanics and Foundations Division, No. SM2, Paper No. 1205.

[5] Hilf, J. W., "A Rapid Method of Construction Control for Embankments of Cohesive Soils," Conference on Soils for Engineering Purposes, ASTM Committee D–18 and Sociedad Mexicana de Mecanica de Suelos, Dec. 9–13, 1957, ASTM Special Technical Publication No. 232, American Society for Testing Materials, Philadelphia, Pa., p. 123.

[6] Davis, F. J., "Quality Control of Earth Embankments," Third International Conference on Soil Mechanics and Foundation Engineering, Switzerland, 1953, vol. 1, p. 218.

[7] Walker, F. C., and Holtz, W. G., "Control of Embankment Material by Laboratory Testing," Trans. ASCE, vol. 118, 1953, p. 1.

[8] Holtz, W. G., and Lowitz, C. A., "Compaction Characteristics of Gravelly Soils," Conference on Soils for Engineering Purposes, ASTM Committee D–18 and Sociedad Mexicana de Mecanica de Suelos, Dec. 9–13, 1957, ASTM Special Technical Publication No. 232, American Society for Testing Materials, Philadelphia, Pa., p. 123.

Example DAM ZONE 1

$D = \dfrac{\text{FILL DRY DENSITY}}{\text{MAXIMUM LABORATORY DRY DENSITY}} \times 100$ (MINUS NO 4 MATERIAL)

	F (PREV)	THIS PERIOD				TO DATE		
		FREQUENCY OF OCCURRENCE	F	CUM F	CUM %	F	CUM F	CUM %
93.0 - 93.9								
94.0 - 94.9								
95.0 - 95.9								
96.0 - 96.9		IIII	4	4	8			
97.0 - 97.9		IIII	4	8	16			
98.0 - 98.9		~~IIII~~ IIII	9	17	34			
99.0 - 99.9		~~IIII~~ II	7	24	48			
100.0 - 100.9		~~IIII~~ III	8	32	64			
101.0 - 101.9		~~IIII~~ ~~IIII~~	10	42	84			
102.0 - 102.9		~~IIII~~	5	47	94			
103.0 - 103.9		III	3	50	100			
104.0 - 104.9								
105.0 - 105.9								
TOTALS			50					

	PREV	THIS PERIOD	TO DATE
Average max lab γ_D (P.C.F.)		117.1	
Average fill γ_D (P.C.F.)		117.1	
Mean variation from max lab γ_D (P.C.F.)		0	
Average rock content (% of plus No 4 by dry weight)		2.6	

PERIOD OF REPORT ______ TO ______
TESTS 9-26-57-0-1R TO 10-24-57-0-1R

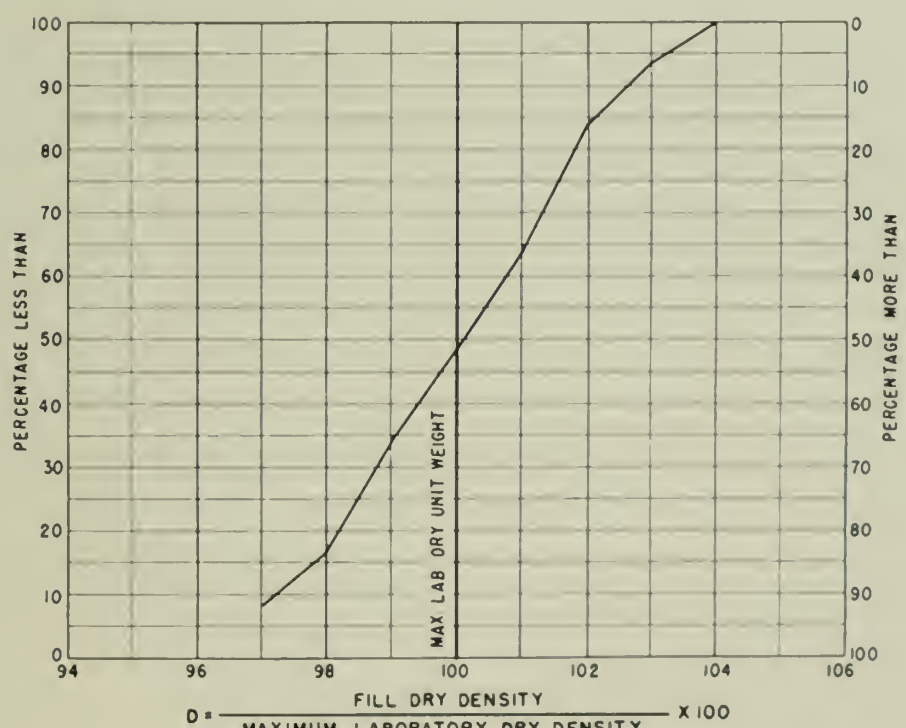


Figure E-11. Statistical analysis of field density tests for density control.

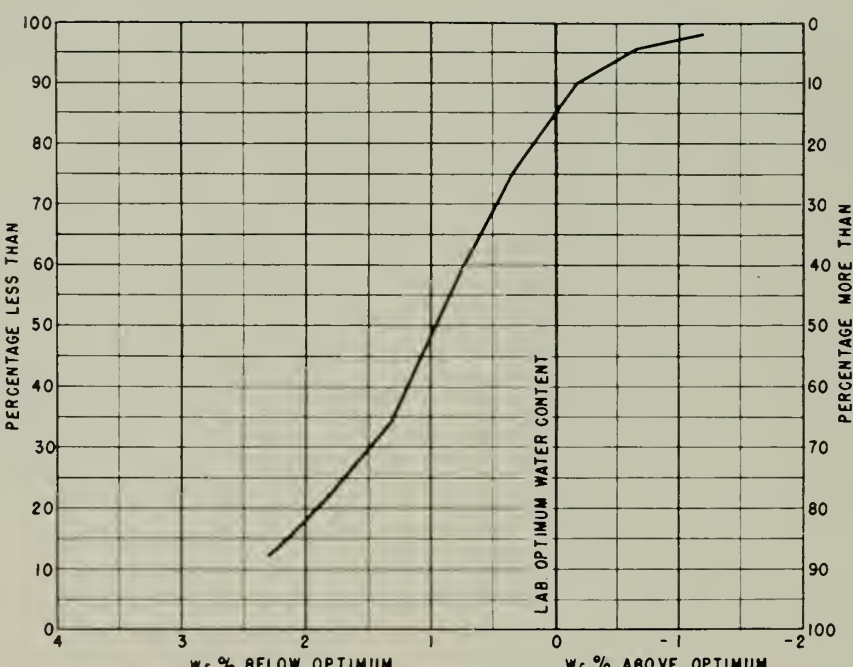
Example DAM ZONE I

OPTIMUM WATER CONTENT MINUS FILL WATER CONTENT ($w_o - w_f$) IN PERCENT OF DRY WEIGHT

		F (PREV.)	THIS PERIOD											TO DATE		
			FREQUENCY OF OCCURRENCE								F	CUM. F	CUM. %	F	CUM. F	CUM. %
w_f IS BELOW OPTIMUM	3.3-3.7															
	2.8-3.2															
	2.3-2.7		卌	I							6	6	12			
	1.8-2.2		卌								5	11	22			
	1.3-1.7		卌	I							6	17	34			
	0.8-1.2		卌	卌	I						11	28	56			
	0.3-0.7		卌	卌							10	38	76			
+0.2 TO -0.2			卌	II							7	45	90			
w_f IS ABOVE OPTIMUM	0.3-0.7		III								3	48	96			
	0.8-1.2		I								1	49	98			
	1.3-1.7															
	1.8-2.2															
	2.3-2.7		I								1	50	100			
	2.8-3.2															
	3.3-3.7															
	TOTALS										50					

	PREV.	THIS PERIOD	TO DATE
Average optimum water content		13.7	
Average fill water content		12.8	
Mean variation from optimum water content		0.9	

PERIOD OF REPORT ________ TO ________
TESTS 9-26-57-a-IR TO 10-24-57-a-IR



OPTIMUM WATER CONTENT MINUS FILL WATER CONTENT ($w_o - w_f$) IN PERCENT OF DRY WEIGHT

Figure E-12. Statistical analysis of field density tests for moisture control.

Concrete in Construction

J. E. BACKSTROM, L. C. PORTER, G. B. WALLACE, AND E. L. ORE [1]

A. CONCRETE AND CONCRETING MATERIALS

F-1. *Important Properties of Concrete.* Concrete is one of the most durable and versatile of all construction materials. It is composed of sand, gravel, crushed rock, or other aggregates held together by a hardened paste of hydraulic cement and water. The selection, testing, and evaluation of these materials, together with their processing and proportioning, form the subject of this appendix. Specifications for concrete are included in appendix G. For complete coverage of concrete as a construction material the reader is referred to the Bureau of Reclamation Concrete Manual [1].[2]

The characteristics of concrete discussed in the following sections should be considered on a relative basis and in terms of the degree of quality that is required for any given construction purpose. In addition to being adequately designed, a structure must be constructed properly of concrete which is strong enough to carry the design loads and which is economical, not only in first cost but also in terms of its ultimate service. In addition to strength, concrete must have the properties of workability and durability.

F-2. *Workability.*—Workability has been defined as the ease with which a given set of materials can be mixed into concrete and subsequently handled, transported, and placed with a minimum loss of homogeneity. Workability is dependent on the proportions of the constituent materials as well as on their individual characteristics. The degree of workability required for proper placement and consolidation of concrete is governed by the dimensions and shape of the structure and by the spacing and size of the reinforcement. For example, concrete having suitable workability for a pavement slab would be difficult or impossible to economically place in a thin, heavily reinforced section. The slump test [2] coupled with judgment developed by experience, is a common means of evaluating concrete workability.

F-3. *Durability.* A durable concrete is one which will withstand, to a satisfactory degree, the effects of service conditions to which it will be subjected, such as weathering, chemical action, and wear.

(a) *Weathering Resistance.*—Disintegration of concrete by weathering is caused mainly by the disruptive action of freezing and thawing and by expansion and contraction, under restraint, resulting from temperature variations and alternate wetting and drying. Concrete can be made that will have excellent resistance to the effects of such exposures if careful attention is given to the selection of materials and to all other phases of job control. The purposeful entrainment of small bubbles of air helps greatly to improve concrete durability. It is also important that, where practicable, provision be made for adequate drainage of exposed concrete surfaces. In general, the more watertight the concrete, the more difficult it is for water to gain entrance and to fill the voids and the greater is the resistance to frost action.

(b) *Resistance to Chemical Deterioration.*—The common causes of chemical deterioration of concrete include: Alkali-aggregate reactivity, in which alkalies in the cement react chemically with mineral constituents of concrete aggregates; deterioration resulting from contact with various chemical agents; and sulfate attack, in which

[1] Engineers, Concrete Laboratory Branch, Bureau of Reclamation.

[2] Numbers in brackets represent items in the bibliography, sec. F-31.

484556 O—60——35

salts (principally soluble sulfates) that are present in ground water or soil in contact with the concrete, attack the cement paste.

Alkali-aggregate reactivity is characterized by the following observable conditions: Cracking, usually of random pattern on a fairly large scale (see fig. F–1); excessive internal and overall expansion; cracks that may be very large at the concrete surfaces (openings up to 1½ inches have been observed), but which extend into the concrete only a distance of from 6 to 18 inches; gelatinous exudations and whitish amorphous deposits, both on the surface and within the mass of the concrete, especially in voids and adjacent to some affected pieces of aggregate; peripheral zones of reactivity, alteration, or infiltration in the aggregate particles, particularly those particles containing opal and certain types of acid and intermediate volcanic rocks; and lifeless chalky appearance of the freshly fractured concrete.

Use of low-alkali cement, that is, cement having a sodium equivalent of 0.6 percent or less as determined by summation of the percentage of sodium oxide plus 0.658 percent of potassium oxide, provides an effective means of preventing harmful alkali-aggregate reaction. Tests to evaluate reactive combinations of aggregate and cement are involved and expensive. Therefore, for jobs with a limited budget, inspection of existing concrete structures near the job site and determination of the source of the aggregate and cement used in the structures may provide valuable information regarding the quality of local materials for use in construction. Also, the need for protective measures frequently can be determined by examination of the prospective aggregate by an experienced petrographer. Should there be any doubt regarding the reactivity of a prospective aggregate after a thorough investigation of the material, maximum safety is assured at little extra expense by specifying low-alkali cement.

Most prominent among aggressive substances which affect concrete structures are the sulfates of

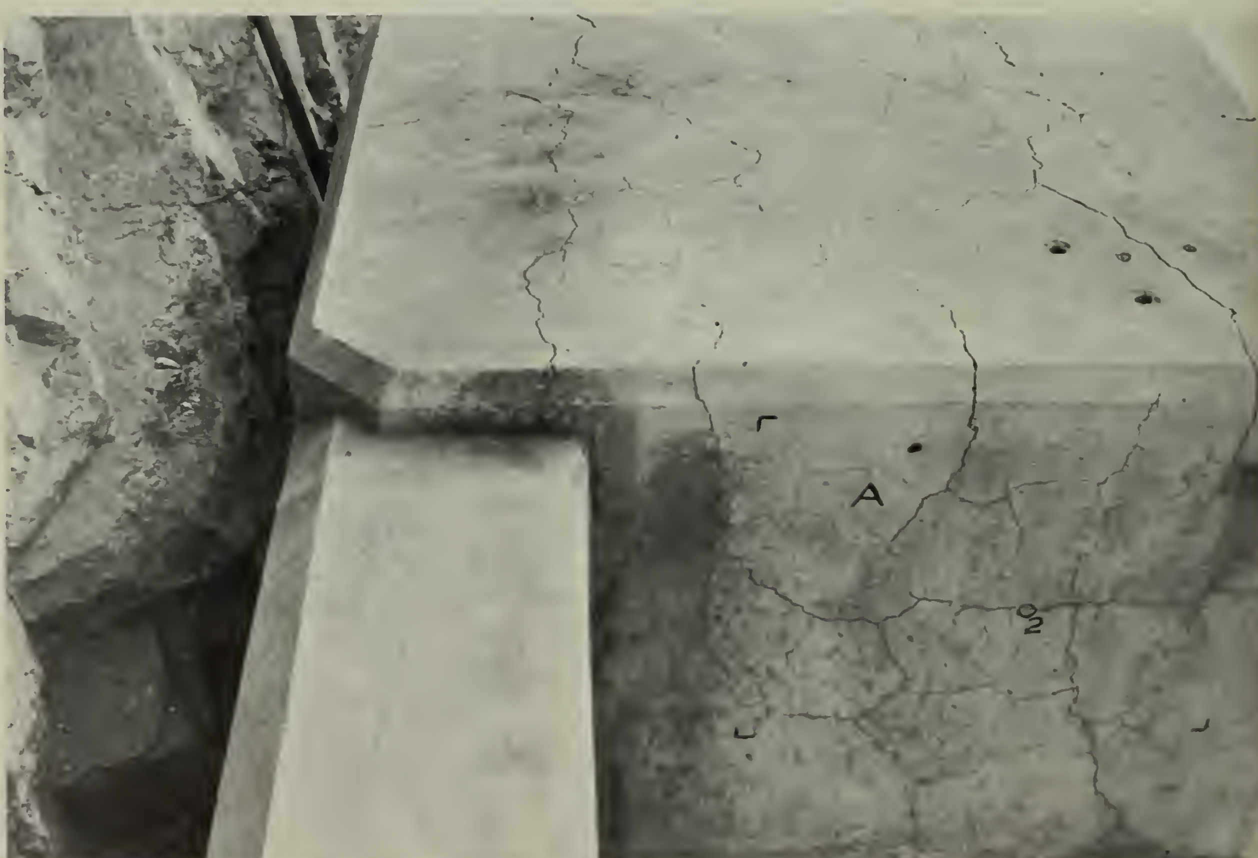


Figure F–1. Typical pattern cracking on the exposed surface of concrete affected by alkali-aggregate reaction.

sodium, magnesium, and calcium. These salts, which are known as "white alkali," are frequently encountered in the "alkali" soils and ground waters of the western half of the United States. The sulfates react chemically with certain compounds in the cement to produce considerable expansion and disruption of the paste. The result of such action is shown in figure F-2. Sulfate attack is reduced by using the type of cement indicated in table F-1 for varying degrees of sulfate concentration. While the type of cement indicated in table F-1 is preferable, reduction in sulfate attack can be obtained through increases in cement content.

Where deposits of "white alkali" occur, it is advisable to examine existing concrete structures in the vicinity of the proposed work to determine whether protection against sulfate attack will be necessary. The presence of these white deposits of salts often indicates the need for testing the soil and ground water to determine if harmful sulfate concentrations are present. Testing is desirable because the white deposits may be *chloride* salts which, compared to sulfate salts, are relatively harmless to hardened concrete.

TABLE F-1 — *Attack on concrete by soils and waters containing various sulfate concentrations*

Relative degree of sulfate attack	Percent water-soluble sulfate (as SO_4) in soil samples	P.p.m. sulfate (as SO_4) in water samples
Negligible	0.00 to 0.10	0 to 150
Positive	0.10 to 0.20	150 to 1,000
Considerable [1]	0.20 to 0.50	1,000 to 2,000
Severe [2]	Over 0.50	Over 2,000

[1] Use type II cement.
[2] Use type V cement.

F-4. *Effects of Curing on Strength* [7].—Experience has demonstrated that when the maximum permissible water-cement ratio has been established on the basis of durability requirements as shown in table F-2, concrete will usually develop adequate compressive strength if properly placed and cured. Figure F-3 shows the compressive strength development of concrete cured for various lengths of time and subsequently stored or dried. Concrete exposed to dry air from the time it is

Figure F-2. Disintegration of concrete caused by sulfate attack.

placed is only about 50 percent as strong at 6 months as concrete moist-cured 14 days before being exposed to dry air.

TABLE F–2.—*Allowable maximum net water-cement ratios for durability of concrete subjected to various degrees of exposure*

Type or location of concrete or structure, and degree of exposure	Water-cement ratio, *W/C*, by weight	
	Severe climate, wide range of temperature, long periods of freezing or frequent freezing and thawing	Mild climate, rainy or arid, rarely snow or frost
A. Concrete in portions of structures subject to exposure of extreme severity, such as the top 2 feet of walls, boxes, piers, and parapets; all of curbs, sills, ledges, copings, corners and cornices; and concrete in the range of fluctuating water levels or spray. These are parts of dams, spillways, wasteways, blowoff boxes, tunnel inlets and outlets, tailrace walls, valve houses, canal structures, and other concrete work.	0. 45±0. 02	0. 55±0. 02
B. Concrete in exposed structures and parts of structures where exposure is less severe than in A, such as portions of tunnel linings and siphons subject to freezing, the exterior of mass concrete, and the other exposed parts of structures not covered by A.	0. 50±0. 02	0. 55±0. 02
C. Concrete in structures or parts of structures to be covered with backfill, or to be continually submerged or otherwise protected from the weather, such as cutoff walls, foundations, and parts of substructures, dams, trashracks, gate chambers, outlet works, and control houses. (If severe exposure during construction appears likely to last several seasons, reduce *W/C* for parts most exposed by 0.05.)	0. 58±0. 02	0. 58±0. 02
D. Concrete that will be subject to attack by sulfate alkalies in soil and ground waters, and will be placed during moderate weather.	--------------	0. 50±0. 02
E. Concrete that will be subject to attack by sulfate alkalies in soil and ground waters, but will be placed during freezing weather, when calcium chloride would normally be used in mix. Do not employ $CaCl_2$, but decrease *W/C* to the value shown.	0. 45±0. 02	------------
F. Concrete deposited by tremie in water------	0. 45±0. 02	0. 45±0. 02
G. Canal lining--------------------------------	0. 53±0. 02	0. 58±0. 02
H. Concrete for the interior of dams------------	The properties of this concrete will be governed by the strength, thermal properties, and volume change requirements which will be established for each structure.	

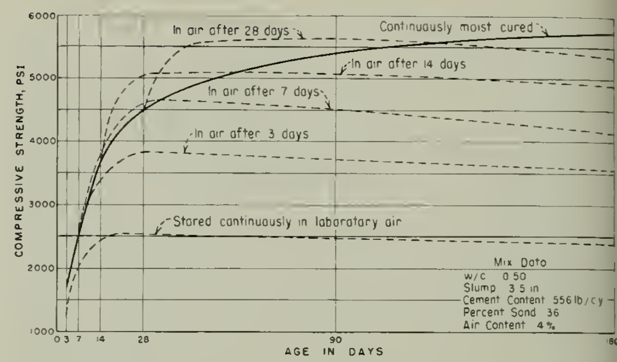


Figure F–3. **Compressive strength of concrete dried in laboratory air after preliminary moist curing.**

F–5. *Effects of Entrained Air on the Properties of Concrete.*—Except for compressive strength, all properties of concrete, including workability, durability, permeability, drying shrinkage, bleeding, etc., are materially improved by the purposeful entrainment of from 2 to 6 percent air, the optimum amount depending on the maximum size aggregate used. Supplementary benefits in the form of reduced water and cement requirements and an increase in ease of finishing may also be realized. Figure F–4 shows the effects of air content on the durability, compressive strength, and required water content of concrete. Note that the durability increases rapidly to a maximum with an addition of air, and then decreases as the air content is further increased, while compressive strength and water content continue to decrease with the increase in air content. Figure F–5 shows the strength in relation to the water-cement ratio for both air-entrained and nonair-entrained concrete. Note that the strength decreases with an increase in water-cement ratio, and that the use of air entrainment also decreases the strength.

F–6. *Types of Portland Cement.*—Because of their size or physical location, structures often require the use of cements having special properties to assure adequate durability and economic life. At the present time there are five main types of cement, which will be briefly discussed. The differences in type are the result of changing the relative proportions of the four predominating chemical compounds.

Type I cement is for use in general concrete construction when the special properties of the other types of cement are not required. This type of cement is suitable for use when there is no exposure to sulfates in the soil or ground water.

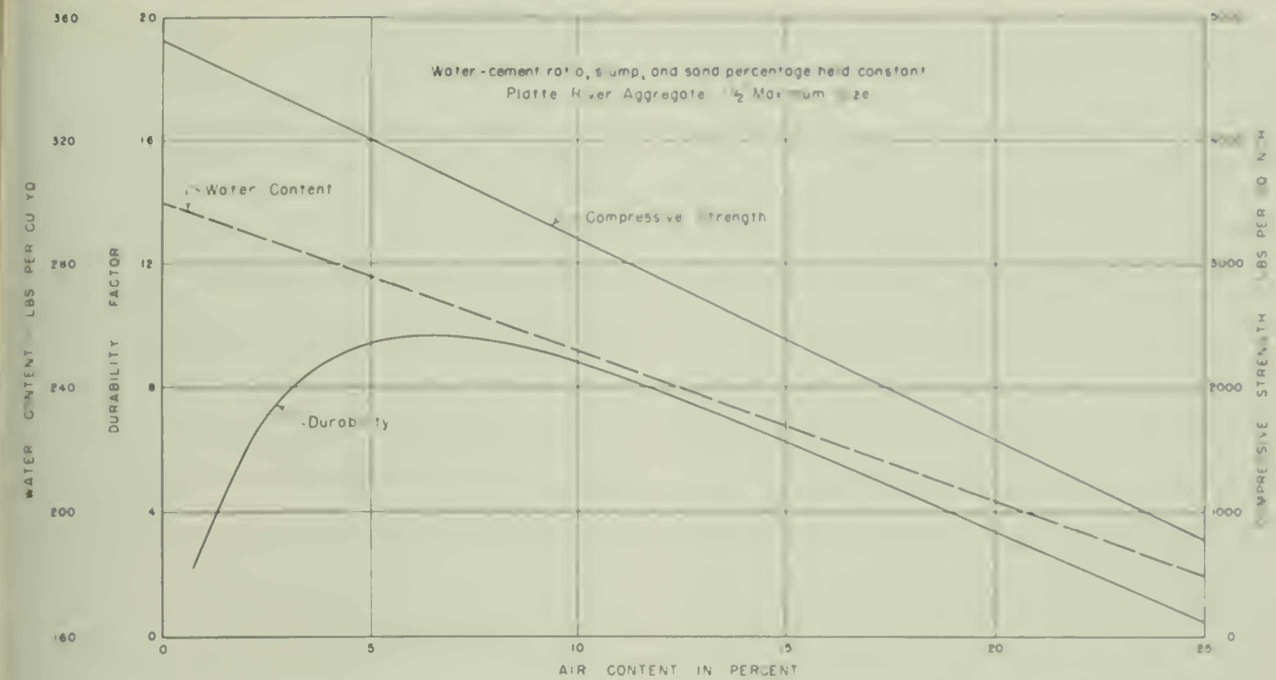


Figure F-4. Effects of air content on durability, compressive strength, and required water content of concrete.

Usually, it is more economical than type II cement.

Type II cement is used where relatively low heat generation is desired or where moderate sulfate attack may occur. Concrete made with type II cement possesses all the good qualities inherent in that containing type I cement.

Type III cement is used where rapid strength development of concrete is essential, as in emergency construction and repairs, and in the construction of machine bases and gate installations. Where this type of cement is used, curing and protection of the concrete may be discontinued at an earlier age.

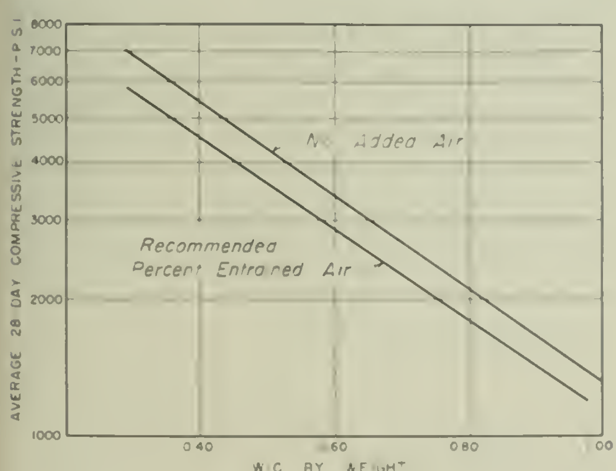


Figure F-5. Strength in relation to water-cement ratio for air-entrained and non-air-entrained concrete.

Type IV cement generates less heat than the other types and at a slower rate. It was developed to reduce the cracking resulting from high temperature rise and subsequent contraction with temperature drop that, in general, accompanies the use of type I or type II cements in massive concrete structures. In addition, concrete in which type IV is used has somewhat greater resistance to sulfate attack than for type I or type II, and has less rapid strength development but equal strength at advanced ages (particularly in the case of mass concrete).

Type V cement is especially beneficial where structures such as canal linings, culverts, and siphons will be in contact with soils and ground waters containing soluble sulfates in such concentrations as would cause serious deterioration of the concrete if other types of cement were used. Concrete containing type V cement is much more resistant to sulfate attack than concretes which contain any of the other types of cement. Compressive strength development, though generally not so rapid, ultimately is approximately equal to that developed by other types.

Any of the five types described above may be purchased to meet the low-alkali provisions of Federal Specification SS–C–192a. Air-entraining cement may also be purchased under these specifications.

F–7. ***False Set in Cement.***—False set in cement (sometimes referred to as premature stiffening) is evidenced in concrete by a significant loss of workability shortly after mixing. It is most likely to prove troublesome where delivery of mixed concrete to the forms is delayed for any cause, as when the batch stands for a few minutes in the mixer or collecting hopper before being discharged. False set may be indicated only by an excessive loss of slump between the mixer and the forms. It may, however, be so severe as to defeat all efforts toward control of uniformity of concrete, to delay construction schedules, and to increase the costs of handling, placing, and finishing operations. Furthermore, false set will increase the water requirement of concrete with resultant lower strength, erratic air-entrainment characteristics, and other adverse effects.

F–8. ***Use of Pozzolans.***—Pozzolans are siliceous or siliceous and aluminous materials, which in themselves possess little or no cementitious value, but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide (lime) at ordinary temperatures to form compounds possessing cementitious properties. The lime required for this reaction is provided by the portland cement. Materials having pozzolanic properties are some clays and shales, volcanic materials (including pumice, pumicite, etc.) and fly ash, a product of some coal-burning boilers.

Pozzolans usually should not be specified for concrete unless there are definite advantages in their use. Pozzolans may be used to improve the workability and quality of concrete, to effect economy, or to protect against disruptive expansion caused by the reaction between certain aggregates and the alkalies in cement. In addition to improving workability of concrete, most pozzolans will reduce heat generation, thermal volume change, bleeding, and permeability of concrete. Compressive strength development is at a slower rate than that of portland cement, but the ultimate strength developed is as great or possibly greater.

F–9. ***Quality and Gradation of Aggregates.***—The procedures for quality and gradation tests for concrete aggregates are outlined in the appendix of the Bureau of Reclamation Concrete Manual [1]. Concrete aggregate usually consists of natural sand and gravel, crushed rock, or mixtures of these materials. Natural sands and gravels are the most common and are used whenever they are of satisfactory quality and can be obtained economically in sufficient quantity. Crushed rock is widely used for coarse aggregate and occasionally is processed to produce sand when suitable materials from natural deposits are not economically available. Production of workable concrete using sharp, angular, crushed fragments usually requires more care and more cement and water than does concrete made with well-rounded sand and gravel. However, the difficulty of making workable concrete with crushed aggregate may be greatly reduced through the extra workability imparted by entrained air.

Aggregate is commonly contaminated by silt, clay, mica, coal, humus, wood fragments, or other organic matter, chemical salts, and surface coatings and encrustations. Some contaminating substances in concrete act in a variety of ways to cause unsoundness, decreased strength and durability, and unsightly appearance; their presence complicates processing and mixing operations. Fortunately, excesses of contaminating substances may frequently be removed by simple washing.

An aggregate is considered to be physically sound if it is adequately strong and is capable of resisting the agencies of weathering without disruption or decomposition. Mineral or rock particles that are physically weak, extremely absorptive, easily cleavable, or which swell when saturated are susceptible to breakdown through exposure to natural weathering processes. The use of such materials in concrete reduces strength or leads to premature deterioration by promoting weak bond between aggregate and cement paste, or by inducing cracking, spalling, or popouts. Shales, friable sandstones, some micaceous rocks, clayey rocks, some very coarsely crystalline rocks, and various cherts are examples of physically unsound aggregate materials.

Chemical soundness of an aggregate is also important. In many instances, excessive expansion causing premature deterioration of concrete has been associated with chemical reaction between reactive aggregate and the alkalies in cement. Known reactive substances are the

silica minerals (opal, chalcedony, tridymite, cristobalite), zeolite, heulandite (and probably ptilolite), glassy to cryptocrystalline rhyolites, dacites and andesites and their tuffs, and certain phyllites.

An aggregate should and usually does have sufficient strength to develop the full strength of the cementing matrix. Generally, resistance of concrete to abrasion is directly related to its *compressive strength* regardless of the type of aggregate employed. Usually, quartz, quartzite, and many dense volcanic and siliceous rocks are well adapted for making strong and, therefore, wear-resistant concrete.

Volume change in aggregate resulting from wetting or drying is a common source of injury to concrete. Shales, clays, and some rock nodules are examples of materials that expand when they absorb water and shrink as they dry.

Flat or elongated particles of aggregate have a detrimental effect on the workability of concrete and require more highly sanded mixes with consequent use of more cement and water. A moderate percentage of flat or elongated fragments in the larger sizes of coarse aggregate has little effect on the workability or cost of concrete.

Specific gravity [3] is a useful, quick indicator of aggregate quality. Low specific gravity frequently indicates porous, weak, and absorptive material, and high specific gravity often indicates good quality. However, such indications are not infallible and should be confirmed by other tests. Specific gravity of aggregate in itself is of direct importance only in those cases where design or structural considerations require that the concrete have minimum or maximum weight. When lightness is desired, artificially prepared aggregates of low unit weight are frequently used in place of natural rock.

The particle-size distribution of aggregate as determined by separation with standard screens is known as its gradation [4]. Aggregate grading is important principally because of its effect on water-cement ratio and paste-aggregate ratio, which affect economy and placeability of concrete. A grading chart, similar to that shown in figure F–6, is useful for depicting the size distribution of the aggregate particles.

F–10. *Quality of Mixing and Curing Water.* Mixing and curing water for concrete should be reasonably clean and free from objectionable quantities of silt, organic matter, alkali, salts, and other impurities. Preparatory to its use in concrete, water from a stream carrying an excessive quantity of suspended solids should be allowed to stand in settling basins or should be clarified by other means. A turbidity limit of 2,000 parts per million is sometimes specified for mixing water. If clear water does not have a saline or brackish taste, it may be used for mixing and curing concrete without further testing. Proposed curing water suspected of containing more than 1,000 parts per million of sulfate should be analyzed.

F–11. *Use of Admixtures.* The early strength of concrete can be materially increased by inclusion of an accelerator such as calcium chloride in the concrete mix. Increased early strength during cold weather affords better protection against damage of concrete from freezing at the end of the specified protection period. Also, high early strengths may be desirable for expediting form removal or to permit early loading of anchor devices.

Use of air entrainment in concrete is a requirement for Bureau of Reclamation construction. Purposeful entrainment of air is accomplished by adding an air-entraining agent to the concrete mix which results in the dispersion, throughout the mix, of noncoalescing spheroids of air having diameters of from approximately 0.003 to 0.05 inches. The amount of air entrained is, in general, a direct function of the quantity of agent added. Among the factors that influence the amount of air entrained in concrete for a given amount of air-entraining agent are: grading and particle shape of aggregate, richness of mix, mixing time, slump, and temperature of concrete.

Desirable air contents, at the mixer, for concrete subject to severe freezing are generally as follows:

Coarse aggregate, maximum size in inches:	*Total air, percent*
$\frac{3}{4}$	6 ± 1
$1\frac{1}{2}$	5 ± 1
3	4 ± 1
6	$3\frac{1}{2} \pm 1$

For concrete not subject to severe freezing, air contents may be reduced as much as one-fourth if strength development is critical and sufficient workability can still be maintained.

Retarders are only recently gaining favor as admixtures for concrete. Generally, these chemicals are added to concrete during hot-weather

SCREEN SIZE	% RETAINED		COMB % RET	
	INDIVIDUAL	CUMULATIVE	INDIVIDUAL	CUMULATIVE
6 INCH	0	0	0	0
3 INCH	28	28	21	21
1½ INCH	26	54	20	41
¾ INCH	22	76	16	57
⅜ INCH	16	92	12	69
No. 4	8	100	6	75
No. 4	0	0	0	
No. 8	12	12	3	78
No. 16	20	32	5	83
No. 30	24	56	6	89
No. 50	24	80	6	95
No. 100	16	96	4	99
PAN	4	100	1	100
F M		2.76		
PERCENT SAND (clean separation) 25				
(Screen sizes are based on square openings)				

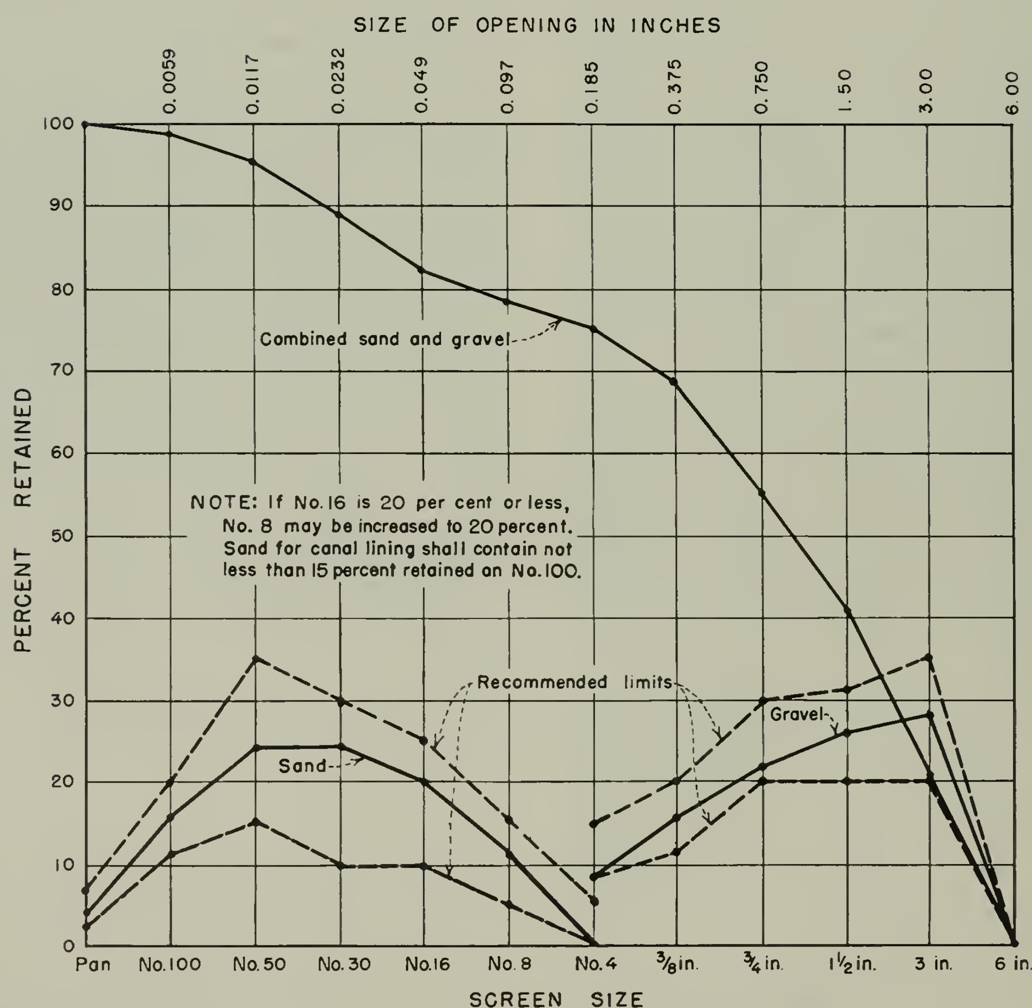


Figure F–6. Typical size distribution of suitably graded natural aggregate.

placing to prevent slump loss, excessive increase in water requirement, and to prolong the workability of concrete during adverse placing conditions. When retarders are used, the manufacturer's recommendations should be followed carefully, as large overdosages of certain of the chemicals may drastically affect the setting time or strength development of the concrete.

F–12. ***Field Control.***—After concrete materials have been selected and the relative proportions determined, their use should be controlled closely. This field control governs the quality, uniformity, and ultimate economy of the concrete structure. Much of the potential value of first-class materials and optimum proportioning may be lost through ineffective control in batching, mixing, handling, placing, and curing. The poorer the quality of the ingredients, the greater the need for rigid control to attain satisfactory durability and strength and, therefore, maximum serviceable life in the structure.

The degree of uniformity of concrete strength is a measure of success in attaining adequate field control. Without adequate control of concrete manufacturing operations, wide variations in strength will occur and extra cement will be needed to insure that the quality of the concrete will meet minimum requirements.

B. DESIGN OF CONCRETE MIXES

F–13. ***Introduction.***—Concrete is composed essentially of water, cement, aggregate, and purposefully entrained air. Proportions of ingredients should be selected to make the most economical use of available materials that will produce concrete of the required workability, durability and strength. Mix proportions should be selected to produce concrete with:

(1) The stiffest consistency (maximum amount of coarse aggregate) that can be placed efficiently with vibration to provide a homogeneous mass;

(2) The maximum size of aggregate economically available and consistent with satisfactory placement by vibration;

(3) Adequate durability to withstand satisfactorily the weather and other destructive agencies to which it may be exposed; and

(4) Sufficient strength to withstand the loads to be imposed without danger of failure.

F–14. ***Estimate of Water Requirement.***—Overwet concrete should always be avoided; it is difficult to place without segregation and it is certain to be weak and lacking in durability. The proper consistency, as determined by the slump test [2], for placing and consolidating concrete in various types of structures is shown in table F–3.

TABLE F–3.—*Recommended maximum slumps for various types of concrete construction*

Type of construction	Maximum slump in inches [1]
Heavy mass construction	2
Canal lining	3
Slabs and tunnel inverts	2
Tops of walls, piers, parapets, and curbs	2
Sidewalls and arch in tunnel lining	4
Other structures	3

[1] These maximum slumps are for concrete after it has been deposited, but before it has been consolidated, and are for mixes having air contents as indicated in table F–4.

The quantity of water per unit volume of concrete required to produce a mix of desired consistency is influenced by the maximum size, particle shape, and grading of the aggregate and by the amount of entrained air. Within the normal range of mixes, the water requirement is relatively unaffected by the quantity of cement. The quantities of water given in table F–4 are of sufficient accuracy for preliminary estimates of proportions. They are the averages that may be expected for various maximum sizes of fairly well shaped and well graded aggregate. Flat-shaped aggregates with excess fines will require more water, and perfectly round shaped, well graded aggregates will not require as much water as shown in table F–4. The weight of water throughout the normal range of placing temperatures may be assumed to be 62.3 pounds per cubic foot.

F–15. ***Estimate of Cement Requirement.***—A fundamental rule for designing plastic concrete mixes is that the strength and durability of hardened concretes, with the same air content, vary inversely with the ratio of the weight of water to the weight of cement. Table F–2 will serve as a guide in selecting maximum permissible water-cement ratios for different severities of exposure when proper use is made of air entrainment.

TABLE F-4.—*Air and water contents for concrete containing natural sand and average coarse aggregate*

FOR FINENESS MODULUS (F.M.)[1] EQUAL TO 2.75 AND A SLUMP OF 2 TO 4 INCHES AT THE MIXER

	Maximum size of coarse aggregate, inches							
	3/8	1/2	3/4	1	1 1/2	2	3	6
Percent dry-rodded unit weight of coarse aggregate per unit volume of concrete	41	52	62	67	73	76	81	87
Recommended air content, percent	8	7	6	5	4.5	4	3.5	3
Average water content, pounds per cubic yard	322	306	283	267	245	229	204	164
Sand, percent of total aggregate by solid volume	59	50	42	37	33	30	28	24

ADJUSTMENT OF VALUES FOR OTHER CONDITIONS

Changes in conditions stipulated	Effect on values		
	Unit water content	Percent sand	Percent of dry-rodded coarse aggregate
Each 0.1 increase or decrease in F.M. of sand		±0.5	∓1
Each 1-inch increase or decrease in slump	±3%		
Each 1 percent increase or decrease in air content	∓3%	(∓)0.5 to 1.0	
For less workable concrete, as in pavements	−8 lb.	−3	+6

[1] The fineness modulus of sand is computed by adding the cumulative percentages retained on the 6 standard screens, from the No. 4 to the No. 100, inclusive, and dividing the sum by 100.

Table F-5 shows an approximation of the minimum strengths to be expected for air-entrained concrete with different water-cement ratios. This table is conservative and can be used in estimating the strength of concrete until verified by tests of compressive strength specimens.

The cement content is calculated, using the lowest maximum permissible water-cement ratio selected from table F-2 or table F-5 and the water requirement from table F-4. The calculation is accomplished by dividing the water requirement by the water-cement ratio. If a minimum cement content is specified, the corresponding water-cement ratio for estimating strength can be computed by dividing the water content by the cement content.

The term "cement" refers to portland cement or a combination of portland cement and pozzolan fully meeting the requirements of applicable Bureau of Reclamation specifications.

TABLE F-5.—*Probable minimum compressive strength of concrete for various water-cement ratios, pounds per square inch*

Water-cement ratio by weight	Compressive strength of air-entrained concrete at 28 days
0.40	4,300
.45	3,900
.50	3,500
.55	3,100
.60	2,700
.65	2,400
.70	2,200

F-16. *Estimate of Entrained Air Requirement.*—Entrainment of air reduces bleeding and segregation, greatly facilitates the handling and placing of concrete, and permits the use of a wider range in aggregate gradation. It makes possible lower sand and water requirements, and also, the curtailed bleeding permits finishing of concrete surfaces earlier, with less work. The most important benefit of entraining air in concrete in severe climates is that it strikingly increases the resistance to the disintegrating action of freezing and thawing. Resistance to chemical attack and permeability is improved by the reduction in capillary and water-channel structure produced by air entrainment. The benefits cited above may be obtained by the recommended percentages of entrained air shown in table F-4. In mild climates, these values may be reduced about one-fourth, provided satisfactory workability can be maintained.

The amount of air-entraining agent required to produce a desired percentage of entrained air varies with the materials used, temperature of the concrete, richness of mix, and consistency of the fresh concrete. Decreasing the slump and increasing temperature and/or cement content of concrete will usually require larger amounts of agent to maintain the desired air content. The manufacturer's recommendation should be used for the initial mix. More or less agent may be added to subsequent mixes as indicated by tests. Tests to determine air content are described in the Bureau of Reclamation Concrete Manual [5]. Agents are marketed in both liquid and powdered forms, but should be added as a solution to the mix water either before the mixer is charged or during charging. Some cements are manufactured with an air-entraining agent integrally blended with the cement. However, if a uniform air

content is to be maintained under varying conditions of temperature, consistency, richness, and materials, the air-entraining agent should be added at the batch plant so that the amount added may be readily adjusted for the changing conditions. Control of the amount of air is necessary for adequately uniform strength of concrete, since an overdose of entrained air will decrease the compressive strength. Cost of air entrainment is very small compared to the benefits received, amounting to only a few cents per cubic yard of concrete.

F-17. *Estimate of Aggregate Requirement.* Larger maximum sizes and quantities of well-graded coarse aggregate require less mortar to fill voids and provide workability. A good estimate of the optimum quantity of a given maximum-size coarse aggregate is shown in table F-4 as the percent of the dry-rodded unit weight per unit volume of concrete. Table F-6 gives the recommended maximum size of aggregate for various types of construction. The largest permissible aggregate size should be used because it permits minimum water and cement requirements. Determination of dry-rodded unit weight of coarse aggregate is described in the Bureau of Reclamation Concrete Manual [6]. Basing the amount of coarse aggregate on a fixed percentage of the dry-rodded unit weight automatically makes allowance for differences in mortar requirements as reflected by void content. For example, angular aggregates have a higher void content and therefore require more mortar than rounded aggregate. The higher void content results in a lower dry-rodded unit weight and therefore decreases the amount of coarse aggregate obtained from the fixed percentage, which automatically produces a greater amount of mortar per unit volume of concrete.

TABLE F-6.—*Maximum sizes of aggregate recommended for various types of construction*

Minimum dimension of section, inches	Maximum size of aggregate,[1] in inches, for: Reinforced walls, beams, and columns	Heavily reinforced slabs	Lightly reinforced or unreinforced slabs
5 or less		$\frac{3}{4}$ to $1\frac{1}{2}$	$\frac{3}{4}$ to $1\frac{1}{2}$
6 to 11	$\frac{3}{4}$ to $1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{2}$ to 3
12 to 29	$1\frac{1}{2}$ to 3	3	3 to 6.
30 or more	$1\frac{1}{2}$ to 3	3	6.

[1] Based on square screen openings.

When the coarse aggregate is estimated as described above, the amount of sand is calculated as follows:

Determine the volume of the known ingredients in cubic feet per cubic yard of concrete. The volume occupied by a material is equal to its weight divided by its density (Density = specific gravity x 62.3). The specific gravity of sand and coarse aggregate may be determined in accordance with the Bureau of Reclamation Concrete Manual [3]. Cement density may be assumed to be 196.2 pounds per cubic foot. The volume of air in cubic feet per cubic yard of concrete equals 27 times the percent air divided by 100. Subtracting the total volume of all ingredients except sand from 27 gives the volume of sand in cubic feet per cubic yard. Multiplying the volume of sand by its density gives its weight in pounds per cubic yard.

F-18. *Computations of Proportions.*—The computations of proportions for concrete mixes can best be explained by means of specific examples. For these examples, the following design criteria and mix materials will be assumed:

(1) Type II cement with a specific gravity of 3.15.

(2) Coarse aggregate with a specific gravity of 2.68.

(3) Sand with a specific gravity of 2.63 and a fineness modulus of 2.75.

(4) Dry-rodded unit weight of coarse aggregate of 105 pounds per cubic foot.

(5) Sufficient air-entraining agent to entrain the percentages of air recommended in table F-4.

Example 1.—The first example involves a reinforced retaining wall, having a minimum thickness of 11 inches. Tables F-3 and F-6 indicate that a 3-inch slump (under "Other structures") and $1\frac{1}{2}$-inch-maximum-size aggregate will be satisfactory. The concrete will be subjected to rather severe climatic exposure and will fall in class B of table F-2. The wall has been designed on the basis of standard 6- by 12-inch cylinders testing greater than 3,000 pounds per square inch at 28 days. In order to insure that all cylinders test over 3,000 pounds per square inch on a well-controlled job, the average strength would have to be over 4,600 pounds per square inch. Experience has shown this criterion to be too conservative and that satisfactory results can be obtained if 80 percent of the tests are above the design strength. On this basis the mix will have to be designed for an

average strength of about 3,450 pounds per square inch. The latter requirement will be used in this example. The computations for example 1 are shown in table F–7.

TABLE F–7.—*Computations of concrete mix proportions—Example 1*

Mix ingredients	Weight, pounds per cubic yard	Conversion of weight to volume	Solid volume, cubic feet per cubic yard
Water: Estimated value from table F–4	245	$\frac{245}{62.3}$	3.93
Cement:			
W/C for durability, class B concrete (table F–2) = 0.50.			
W/C for strength (table F–5) = 0.51 (durability controls, therefore use 0.50).			
Cement $= \frac{\text{water content}}{W/C} = \frac{245}{0.50} =$	490	$\frac{490}{3.15\times 62.3}$	2.50
Air: From table F–4, 4.5 percent: $0.045\times 27=$			1.21
Coarse aggregate:			
Percent of dry-rodded unit weight from table F–4 = 73 percent.			
Dry-rodded unit weight = 105 pounds × 0.73 = 76.6 pounds per cubic foot × 27 =	2,070	$\frac{2,070}{2.68\times 62.3}$	12.40
All ingredients except sand	2,805		20.04
Sand:			
Volume 27 − 20.04 =			6.96
Weight 6.96 × 2.63 × 62.3 =	1,140		
Total	3,945		27.00

F–19. *Alternate Method of Computations of Proportions.*

—If it is necessary to estimate the mix proportions without determining the dry-rodded unit weight of coarse aggregate, the percentages of sand shown in table F–4 may be utilized. The total volume of aggregate in cubic feet per cubic yard of concrete (both sand and coarse aggregate) is computed by subtracting from 27 the cubic feet of water, cement, and air per cubic yard of concrete. The volume of sand in cubic feet per cubic yard of concrete is obtained by multiplying the cubic feet of total aggregate in a cubic yard of concrete by the percentage of sand extracted from table F–4.

Computations utilizing this alternate method are illustrated in the following example. For this example, the design criteria and mix materials will be the same as for example 1, except that the dry-rodded unit weight is unknown.

Example 2.—The second example (table F–8) requires a concrete mix for a powerhouse foundation which will not be exposed to freezing and thawing. This condition will permit the use of class C concrete from table F–2. The designers have specified a design strength of 2,870 pounds per square inch at 28 days. Tables F–3 and F–6 indicate that a 3-inch slump (again under "Other structures") and 3-inch-maximum-size aggregate will be satisfactory.

F–20. *Batch Weights for Field Use.*

—The preceding trial-mix computations provide the batch quantities for a cubic yard of concrete. It is seldom possible to mix concrete in exactly 1-cubic-yard batches. Therefore, these quantities must be converted in proportion to the size of batch to be used. This conversion can be accomplished by multiplying the 1-cubic-yard quantity of each ingredient by the volume of the new batch in cubic yards. This volume can readily be computed from the proportions of any one of the ingredients in the 1-cubic-yard batch and the new batch. For example, assume that a 3-sack mixer is available and that the trial mix in example 1 is to be used. The volume of the new batch is three times the weight of a sack of cement, divided

TABLE F–8.—*Alternate method of computations for concrete mix proportions—Example 2*

Mix ingredients	Weight, pounds per cubic yard	Conversion of weight to volume	Solid volume, cubic feet per cubic yard
Water: Estimated value from table F–4	204	$\frac{204}{62.3}$	3.27
Cement:			
W/C for durability, class C concrete (table F–2) = 0.58.			
W/C for strength (table F–5) = 0.58.			
Cementing materials $= \frac{\text{water content}}{W/C} = \frac{204}{0.58} =$ 352 pounds	352	$\frac{352}{3.15\times 62.3}$	1.79
Air: From table F–4, 3.5 percent: $0.035\times 27=$			0.95
All ingredients except aggregate	556		6.01
Aggregate:			
Volume = 27 − 6.01 =			20.99
Percent sand (table F–4) = 28 percent.			
Volume of sand = 0.28 × 20.99 =			5.88
Volume of coarse aggregate = 20.99 − 5.88 =			15.11
Weight of sand = 5.88 × 2.63 × 62.3 =	963		
Weight of coarse aggregate = 15.11 × 2.68 × 62.3 =	2,523		
Total	4,042		27.00

by the weight of cement for the 1-cubic-yard batch, or

$$\frac{3\times 94}{490}=\frac{282}{490}=0.575 \text{ cubic yard.}$$

The field batch proportions will be as follows:

Ingredient	*Weight, pounds*	*Volume, cubic feet*
Water	0.575×245=141	0.575×3.93=2.26
Cement (3 sacks)	0.575×490=282	0.575×2.50=1.44
Sand	0.575×1,140=655	0.575×6.96=4.00
Coarse aggregate	0.575×2,070=1,190	0.575×12.40=7.13

The aggregates have been assumed to be in a saturated surface-dry state. Under field conditions, they will generally be moist, and the quantities to be batched in the mixer must be adjusted accordingly. Assume that tests show the sand to contain 5 percent and the coarse aggregate 1 percent free moisture. Since the quantity of saturated surface-dry sand required was 655 pounds, the amount of moist sand to be weighed is 655×1.05=688 pounds. Similarly, the weight of moist coarse aggregate is 1,190×1.01=1,202 pounds. Coarse aggregate is sometimes drier than saturated surface-dry. Assuming an absorption of 1 percent, the amount of saturated surface-dry aggregate required would be 1,190×0.99=1,178 pounds.

The free water in the aggregate must be considered as part of the mixing water, whereas water must be added to allow for absorption in the case of dry aggregate. In our example, free water (mixing water) in the sand is 688−655=33 pounds, and in the coarse aggregate 1,202−1,190=12 pounds. If the coarse aggregate were dry, 1,190−1,178=12 pounds of water must be added to the mixing water to allow for absorption.

F-21. ***Adjustment of Trial Mix.*** In example 1, assume that after a few batches of concrete have been mixed on the job, 150 pounds of water are found to be needed to produce a 3-inch slump instead of the estimated 141 pounds.

The new water-cement ratio is $\frac{150}{282}=0.53$, by weight. The adjusted batch weights and volumes are shown in table F-9. The actual weight of each ingredient in pounds per cubic yard is equal to the batch weight of the ingredient divided by the batch volume in cubic yards. The unit weight test shows the concrete to weigh 147.4 pounds per cubic foot.

$$\text{Batch volume}=\frac{\text{summation of batch weights}}{\text{measured unit weight}\times 27}=\frac{2,277}{147.4\times 27}=0.572 \text{ cubic yard}$$

$$\text{Water content}=\frac{150}{0.572}=262 \text{ pounds per cubic yard}$$

$$\text{Cement content}=\frac{282}{0.572}=493 \text{ pounds per cubic yard}$$

$$\text{Sand content}=\frac{655}{0.572}=1,145 \text{ pounds per cubic yard}$$

$$\text{Coarse aggregate content}=\frac{1,190}{0.572}=2,080 \text{ pounds per cubic yard.}$$

F-22. ***Air-Entraining Agent.***—The percentage of entrained air in the concrete can be determined from the unit weight and pressure air meter [5]. It is advantageous to utilize both methods, as any appreciable difference in results indicates an error and may lead to the discovery of mistakes in mix design computations or in the test methods.

TABLE F-9.—*Adjusted batch weights and volumes*

Ingredients	Weights, pounds per batch	Cubic feet per batch
Water	150	$\frac{150}{62.3}=2.41$
Cement [1]	282	1.44
Sand [1]	655	4.00
Gravel [1]	1,190	7.13
Total	2,277	14.98

[1] Same weight and volume as original batch.

Using the measured unit weight, the actual volume of the batch in example 1 was calculated to be 0.572 cubic yard, which is equal to 15.45 cubic feet. The volume of air in the batch is determined by subtracting the volume of the weighed ingredients from the total volume of the batch. The percentage of entrained air is calculated by dividing

the volume of air by the volume of the batch and multiplying by 100, or

$$\text{Percent air}=\frac{(15.45-14.98)}{15.45}\ 100=3.0.$$

This deficiency in air content accounts for the high water requirement and the slightly greater weight of ingredients per cubic yard as compared with the unadjusted trial mix. The amount of agent necessary to produce 4½ percent air at the same 3-inch slump will be approximately equal to $\frac{4.5}{3}$, or 150 percent of that used in the first trial. Table F–4 indicates that an increase of 1½ percent of entrained air will allow a decrease in water requirement of about 4 percent to maintain a 3-inch slump. The corrected unit water content equals $262-(262\times 0.04)=251$ pounds per cubic yard, which value is close to the original estimate for 4½ percent air.

The corrected value for air-entraining agent should result in concrete having the desired 3-inch slump and 4½ percent air. Figure F–7 illustrates a convenient form for recording trial mix data. The data presented conform to the trial mix in the preceding example 1.

The job mix should not be adjusted for minor fluctuations in water-cement ratio. A variation in water-cement ratio of plus or minus 0.02, by weight, resulting from maintenance of constant slump, is considered normal.

F–23. *Mixes for Small Jobs.*—For small jobs, where time and personnel are not available to determine the proportions in accordance with the recommended procedure, mixes in table F–10 will provide concrete that is amply strong and durable if the amount of water added at the mixer is never large enough to make the concrete overwet. These mixes have been predetermined in conformity with recommended procedure by assuming conditions applicable to the average small job, and for aggregate of average specific gravity. Three mixes are given for each maximum size of coarse aggregate. Table F–6 may be used as a guide in selecting an appropriate maximum size of aggregate. Mix B for each size of coarse aggregate is intended for use

CONCRETE MIX COMPUTATION SHEET

SERIES NO. TX DATE CAST 3-20-53

	PAN	*100	*50	*30	*16	*8	
SAND GRADING %	3	15	29	24	15	14	FM= 2.75

	*4-3/8"	3/8-3/4"	3/4-1½"	1½-3"	3-6"	D.R. UNIT WT—LB/CU FT.
C. AGG. GRADING %	20	30	50			

TARE UNIT WT. CAN 59.5

INGREDIENTS	NAME AND LAB NO.		% ABS: % MOIST.	SP. G.	DENSITY LBS./CU. FT.
WATER	Tap			1	62.3
CEMENT	Lehigh	6742		3.15	196.2
POZZOLAN	-			2.50	155.8
SAND	Clear Creek	8875	5.0	2.63	163.8
C. AGG.	Clear Creek	8876	1.0	2.68	167.0

AGENT Dresinate xx % SOLIDS Dry

TRIAL MIX COMPUTATIONS

MIX NO.	$\frac{W}{C+P}$ BY WT	% SAND OR % DRY RODDED C AGG.	ESTIMATED QUANTITIES PER CUBIC YARD: AGENT		AIR	WATER	CEMENT	POZZ.	SAND	C AGG	TOTAL
Example			15	LB		245	490	-	1140	2070	3945
1	0.50	73	g	CU FT.	1.21	3.93	2.50	-	6.96	12.40	27.00
				LB							
				CU FT.							
				LB							
				CU FT.							
				LB							
				CU. FT.							
				LB.							
				CU. FT.							
				LB							
				CU. FT.							

BATCH COMPUTATIONS

VOL. OF BATCH IN CU YD.	QUANTITIES PER BATCH: AGENT	WATER	CEMENT	POZZ.	SAT SUR DRY: SAND	SAT SUR DRY: C AGG	TOTAL	ADJUSTMENT FOR MOISTURE: ADJUSTED WEIGHT: SAND	ADJUSTED WEIGHT: C AGG	MIXING WATER: IN SAND	IN C AGG	ADDED	TOTAL
0.575	8.6 g	141	282	-	655	1190	2268	688	1202	33	12	105	150

RESULTS OF TESTS ON FRESH CONCRETE

MIX NO.	TEMP °F	WORKABILITY: SLUMP IN.	REMOLDING NO. DROPS	FINISH	APPEARANCE	WT. OF CAN + CONC LBS.	UNIT WEIGHT LBS./CU. FT.	VOL. BATCH CU FT / CU YD*	ACTUAL QUANTITIES PER CUBIC YARD: AGENT		WATER	CEMENT	POZZ.	SAND	C AGG	TOTAL	% AIR: GRAV	PRESS.	$\frac{W}{C+P}$ BY WT.	SPECIMEN NO. AND NOTES
Example									15.0	LB.	262	493	-	1145	2080	3980				
1		3.0	25	Good	Good	206.9	147.4	0.572	g	CU. FT.	4.19	2.51	-	6.99	12.46	26.15	3.1	3.0	0.53	
										LB										
										CU FT.										
										LB.										
										CU FT.										
										LB.										
										CU FT.										
										LB										
										CU FT.										
										LB.										
										CU. FT.										

*OR BATCHES PER CU. YD.

Figure F–7. **Typical trial computations for concrete mix.**

as a starting mix in table F–10. If this mix is undersanded, change to mix A, or, if it is oversanded, change to mix C. Note that the mixes listed in the table apply where the sand is dry. If the sand is moist or very wet, make the corrections in batch weight prescribed in the note.

The approximate cement content in bags per cubic yard of concrete listed in the table will be helpful in estimating cement requirements for the job. These requirements are based on concrete that has just enough water in it to permit ready working into the forms without objectionable separation. Concrete should slide, not run, off a shovel.

C. MANUFACTURE, PLACEMENT, CURING, AND INSPECTION OF CONCRETE

F–24. ***Aggregate Production and Control.*** The control of production and handling of concrete aggregates is often complicated by lack of uniformity in sources of supply and difficulty in maintaining uniformity in the finished production. It is a problem that requires the constant vigilance of the construction engineer. Deleterious materials are ordinarily removed by washing. Unsatisfactory grading requires corrections by wasting of surplus sizes or by supplying deficiencies, or both. Breakage must be minimized and the moisture content of the aggregate should be kept as uniform as practicable.

The gradation of sand as it comes from the pit often does not conform to the specifications and some form of processing is required. Defects in grading may be corrected by adding suitable blending sand, by crushing a portion of the excess of larger sizes, by removing portions of sizes present in excessive amounts, or by a combination of methods. Wet processing is more prevalent than dry processing for this purpose, because sand is commonly damp as it is excavated from the deposit.

Use of sand manufactured by crushing or grinding rock or gravel may result in a harsh mix and should be resorted to only when it is not practicable to obtain suitable natural sand at reasonable cost. Since the angular shape of crushed sand is its only disadvantage, it is important that crushing machines and equipment be used which will produce the best shape of particles from the material to be crushed. Sand produced by crushing in rollers is generally unsatisfactory because of the high percentage of thin and elongated particles. The product of a rod mill is much better in this respect. If the material is not too hard, as in the case of limestone, good results may be obtained with equipment of the impact type, more commonly known as the hammer mill, which excels in producing particles that approach a cubicle shape.

TABLE F–10.—*Concrete mixes for small jobs* [1]

Maximum size of aggregate, inches	Mix designation	Approximate bags cement per cubic yard of concrete	Pounds of aggregate per 1-bag batch		
			Sand [2]		Gravel or crushed stone
			Air-entrained concrete [3]	Concrete without air	
½	A	7.0	235	245	170
	B	6.9	225	235	190
	C	6.8	225	235	205
¾	A	6.6	225	235	225
	B	6.4	225	235	245
	C	6.3	215	225	265
1	A	6.4	225	235	245
	B	6.2	215	225	275
	C	6.1	205	215	290
1½	A	6.0	225	235	290
	B	5.8	215	225	320
	C	5.7	205	215	345
2	A	5.7	225	235	330
	B	5.6	215	225	360
	C	5.4	205	215	380

[1] Procedure: Select the proper maximum size of aggregate. Then, using mix B, add just enough water to produce a sufficiently workable consistency If the concrete appears to be undersanded, use mix A, and if it appears to be oversanded, use mix C.

[2] Weights are for dry sand. If damp sand is used, increase the weight of sand 10 pounds for a 1-bag batch; and if very wet sand is used, add 20 pounds for a 1-bag batch.

[3] Air-entrained concrete is specified for all Bureau of Reclamation work. In general, air-entrained concrete should be used in all structures that will be exposed to alternate cycles of freezing and thawing.

Natural river gravels are usually well shaped by stream action, and satisfactory coarse aggregate with the desired grading can be produced with a minimum of plant equipment. However, in some cases where natural coarse aggregate is not economically available, crushed aggregate is used. Although the shape of the individual particles is important, it is not so critical for coarse aggregate

as it is for sand. Use of corrugated roll crushers to produce smaller sizes of coarse aggregate and of gyratory crushers to produce the large sizes generally results in the least amount of flat and elongated pieces. Because of the segregation and breakage which results, handling should be kept to a minimum during stockpiling operations. Figure F–8 shows correct and incorrect methods of stockpiling.

Since some breakdown of materials will occur regardless of the care exercised in stockpiling, it is desirable to finish screen coarse aggregates at the batch plant to assure production of uniform concrete. It is recommended that this requirement be specified when the quantity of concrete exceeds 10,000 cubic yards.

Periodic analysis of aggregate materials should be made to determine the specific gravity and moisture content of the aggregate, and to determine the relative percentages of the various size fractions. The frequency of these tests should be sufficient to assure that the aggregates are in accordance with specifications requirements.

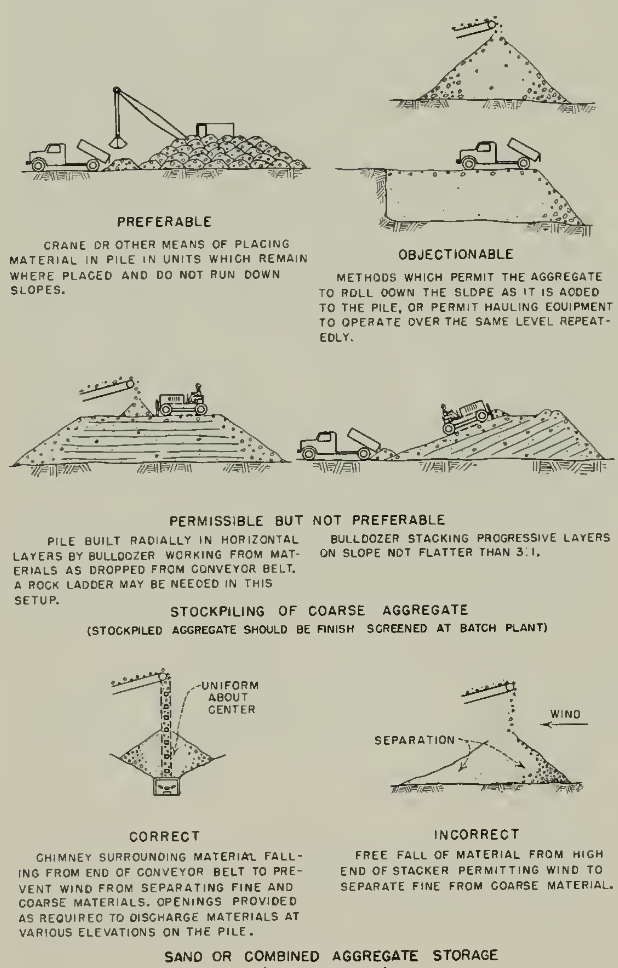


Figure F–8. **Methods of stockpiling aggregates.**

F–25. *Batching Methods and Facilities at Concrete Mixing Plants.*—In order that full advantage of accurate weight batching may be realized, the weighed materials must be properly and carefully handled to the end that batches reaching the mixer will be uniform and complete when released by the measuring equipment.

Tilting mixers are generally more efficient than other types because they can be discharged quickly with a minimum of segregation. Regardless of the type of mixer, to maintain efficiency the mixing blades should be properly spaced, inspected frequently, and repaired when worn, and the interior of the drum should be kept clean and free of deposits of hardened concrete or mortar.

More attention and effort are usually required to obtain uniform slump and mix proportions at minimum water content from truck mixers than from stationary mixers. There is often considerable slump loss in truck-mixed concrete, especially in warm weather. Such loss can be kept to a minimum by stopping initial mixing at about 30 revolutions and by avoiding overmixing. Other precautions which can be taken in warm weather are as follows:

(1) Mixer drums should be painted white and kept white.

(2) Materials should be kept as cool as practicable by shading and by light spraying to promote evaporative cooling.

(3) Water should be as cold as practicable and kept cold by shading and by painting tanks and surface lines white.

(4) Delays prior to the discharge and placement of the concrete should be avoided by organizing the work for prompt handling.

F–26. *Production of Quality Concrete.*—The assurance of uniform and economical concrete is largely dependent on inspection at the batching and mixing plants. Mix adjustments are made, using results of gradation and moisture tests of aggregates. Fresh concrete is tested for consistency, temperature, air content, and unit weight, and concrete cylinders are made for compressive strength tests. The frequency of sampling and testing the concrete will vary with the type and size of job. In general, it will be suffi-

cient to sample and test each class of concrete once during each shift. The samples should be representative of materials used and concrete placed during that shift.

F–27. ***Preparations Preliminary to Placing.***—Before concrete is ordered for placing, adequate inspection should be performed to insure that: (1) Foundations are properly prepared and ready to receive the concrete, (2) construction joints are free from defective concrete and clean, (3) forms are grout-tight, amply strong, and set to line and grade, (4) all reinforcement steel and embedded parts are clean, in their correct position, and securely held in place, and (5) adequate concreting equipment and facilities are on the job, ready to go, and capable of completing the placement without additional unplanned construction joints. Detailed requirements for these items are given in the Concrete Specifications portion of appendix G.

F–28. ***Transporting.***—Even though concrete is carefully designed and properly mixed, its quality may be seriously impaired by use of improper or careless methods in transporting and placing. Buckets, when designed for the job conditions and properly operated, are a satisfactory means for handling and placing concrete. They are not, however, to be used where they have to be hauled so far by truck or railroad that there will be noticeable separation or bleeding due to settlement, or there will be a loss of slump greater than 1 inch.

Dumpcrete trucks are convenient for the distribution of concrete from a central mixer to small and medium size structures. Care must be taken to avoid segregation during the filling and discharging of these units. No free water should be on the surface of the concrete as delivered, nor should there be an objectionable amount of settlement of coarse aggregate or caking at the bottom of the load. Such stratification or settlement can be reduced considerably by use of agitator bodies mounted on trucks, or preferably by mixing the concrete near the point of placement in portable mixers which are supplied by dry-batch trucks.

As ordinarily used, chutes are unsatisfactory devices for transporting concrete because they result in objectionable segregation and slump loss. To avoid these conditions, the following requirements must be fulfilled:

(1) The chute must be on a slope sufficiently steep to handle concrete of the least slump that can be worked and vibrated, and must be supported so that the slope will be constant for varying loads.

(2) If more than a few feet long, the chute must be protected from wind and sun in order to prevent slump loss.

(3) Effective end control that will produce a vertical drop and prevent separation of the concrete ingredients must be provided, preferably in the form of two sections of metal drop chutes, as shown in figure F–9.

(4) With pneumatic methods, separation of coarse aggregate will result from the impact of violently discharged concrete unless the

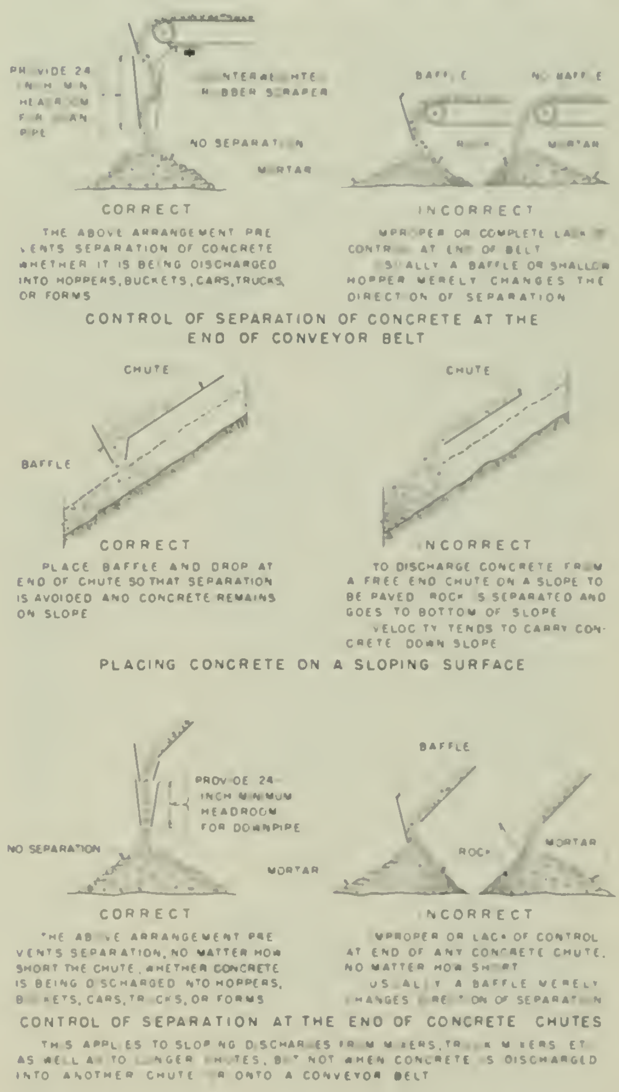


Figure F–9. **Methods of handling concrete at ends of conveyors and chutes.**

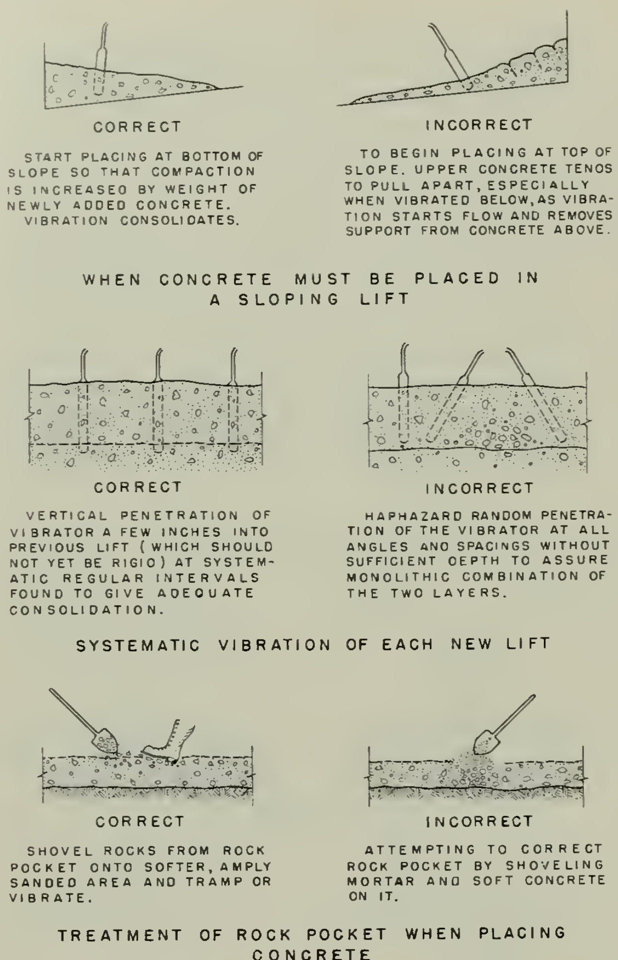


***Figure F-10.* Methods of vibrating and working of concrete.**

end of the discharge line is always buried in fresh concrete. Specifications should therefore require that pneumatic equipment used in placing concrete be such as to permit introduction of the concrete into the forms without high-velocity discharge. A further objection to the pneumatic method is a loss of slump which occurs in the shooting process. Slump losses as great as 3½ inches between mixer and forms have been observed and a loss of 2 to 3 inches is not uncommon.

Pumping through steel pipelines is one of the most satisfactory methods of transporting concrete where space is limited, such as in tunnels, bridge decks, powerhouses, and buildings. Although pump lines longer than 1,000 feet are not recommended, concrete has been pumped through straight, horizontal pipe under the most favorable conditions as far as 1,300 feet. Curves, lifts, and harsh concrete material reduce the maximum pumping distance. For example, a 90° bend is equivalent to about 40 feet of straight, horizontal line, and each foot of head is equivalent to about 8 feet of line. Although manufacturers rate their largest equipment as capable of handling concrete containing aggregate up to 3 inches in size, experience indicates that operating difficulties will be materially lessened if the maximum size of aggregate pumped through such equipment is limited to 2½ or 2¾ inches. A pump will make good progress handling concrete with a slump of 3 to 4 inches and containing 2 to 3 percent more sand than required for concrete to be transported and

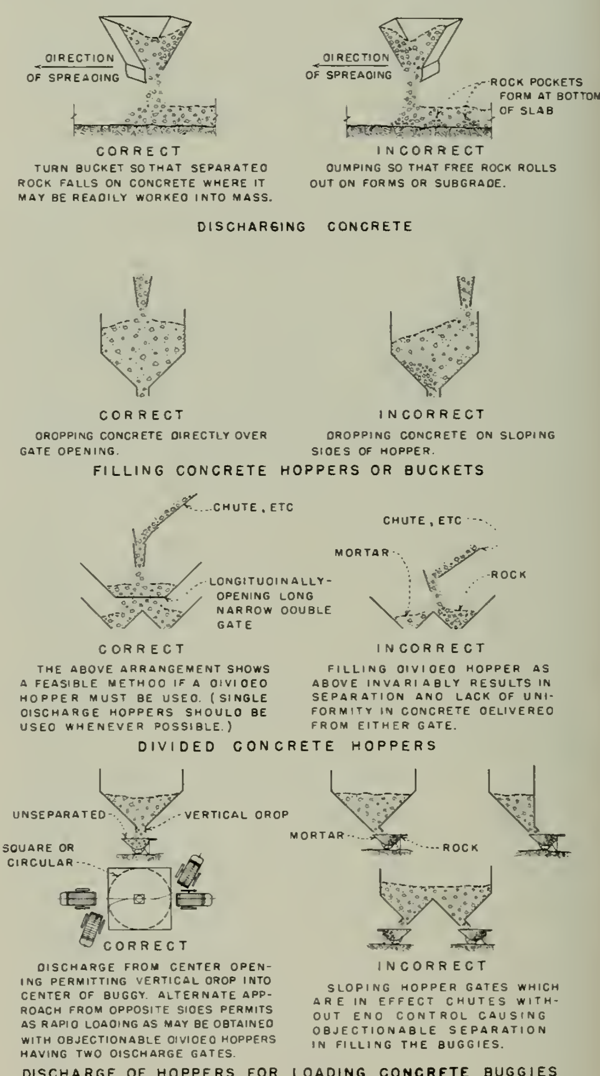


***Figure F-11.* Methods of handling concrete with buckets, hoppers, and buggies.**

placed by gravity methods. Normal rated capacities range from 15 to 65 cubic yards per hour.

F-29. ***Placing.*** Properly placed concrete is free of segregation and the mortar is intimately in contact with the coarse aggregate, the reinforcement and other embedded parts. If any one detail of the placing inspector's many duties deserves special emphasis, it is that of guarding against objectionable segregation during concrete placement. Separation of coarse aggregate from the mortar may be minimized by avoiding or controlling the lateral movement of concrete during handling and placing operations as illustrated in figures F–9, F–10, and F–11. The concrete should be deposited as nearly as practicable in its final position. Placing methods which cause the concrete to flow in the forms should be avoided. These methods result in concentrations of less durable mortar in the ends of walls and corners where durability is most important and encourage the use of a mix that is wetter than necessary. The concrete should be placed in horizontal layers and each layer thoroughly vibrated. Practicable depths of layers for structural concrete range from 12 to 20 inches.

Hoppers for drop chutes should have throat openings of sufficient area to readily pass concrete of the lowest slump that is practicable to work and vibrate. If drop chutes are discharged directly through form ports, considerable separation results and rock pockets and honeycombs will probably be formed. Provision for an outside pocket below each port as shown in figure F–12 will check the fall of the concrete and permit it to flow in the form with a minimum of separation.

Concrete for the top 2 feet of walls, piers, and columns should be of the lowest slump that can be adequately vibrated. After initial vibration, the concrete should be left for 1 or 2 hours to settle and complete the bleeding process. The surfaces should then be topped off with additional concrete as required and the top 2 feet revibrated to close bleeding channels.

When placing an unformed slab on a slope, there is a tendency to place the concrete using a stiff mix that will not slough. Drill cores have shown that the placement of such low-slump concrete without thorough vibration usually results in considerable honeycombing on the underside. To avoid such results, the consistency for this purpose should not be stiffer than a $2\frac{1}{2}$-inch slump. Concrete with this consistency will barely stay on the slope, but it should not be drier. After spreading, the concrete should be thoroughly and systematically vibrated, preferably just ahead of a weighted

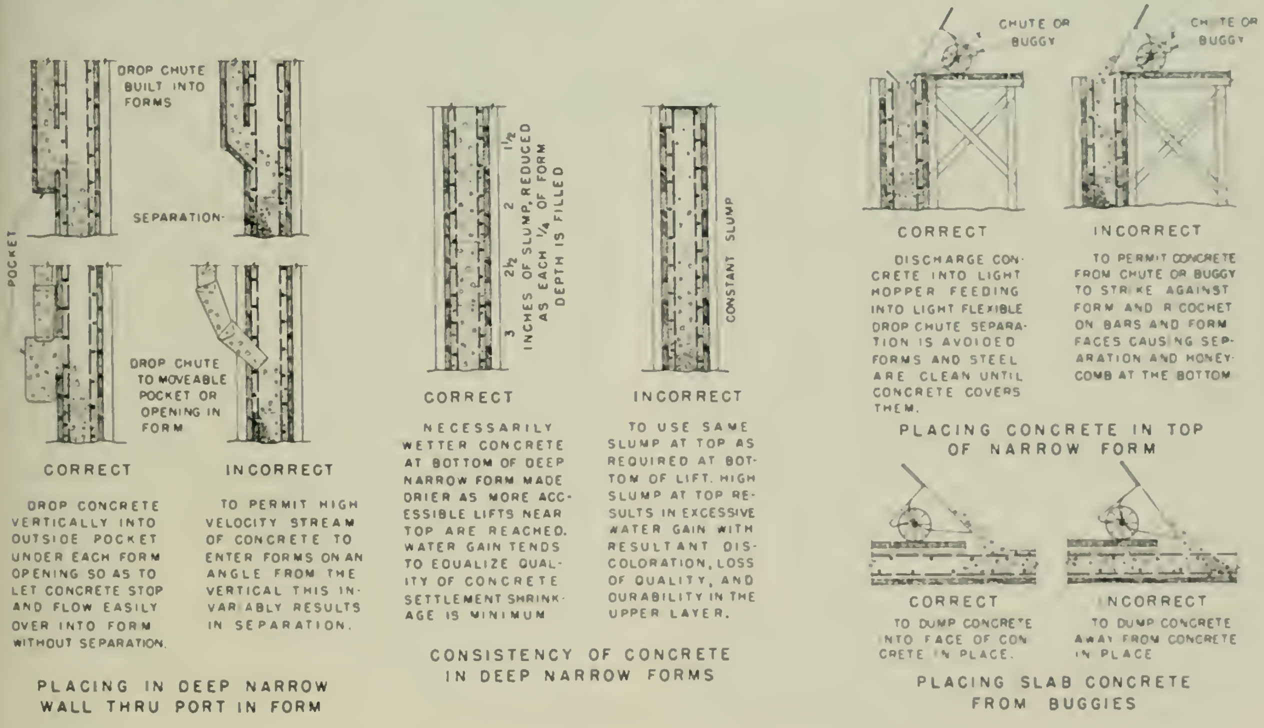


Figure F–12. **Placing concrete in forms.**

steel-faced slipform screed working up the slope as shown in figure F–13.

F–30. *Curing.*—Early drying must be prevented or concrete will not reach its full potential quality. In warm, dry, windy weather, corners, edges, and surfaces become dry very quickly. If these portions are prevented from drying so as to fully develop their hardness, it is certain that interior portions of the concrete will be adequately cured. Wet burlap in contact with the concrete is excellent for curing purposes. It not only shades the concrete, but it also holds the moisture needed for good curing. Wood forms left in place furnish good protection from the sun, but will not keep the concrete sufficiently moist for good curing of outdoor concrete. There is no better curing than that provided by well-moistened backfill. Ponding of floors, pavement, and other slabs also provides excellent curing and reduces crazing, cracking, and wear.

Where water is not economically available, it is often desirable to cure concrete by applying to the exposed surfaces, immediately after form removal, a sealing compound designed to restrict evaporation of the mixing water. An effective compound, properly applied, will, under most conditions, retain enough moisture for adequate curing.

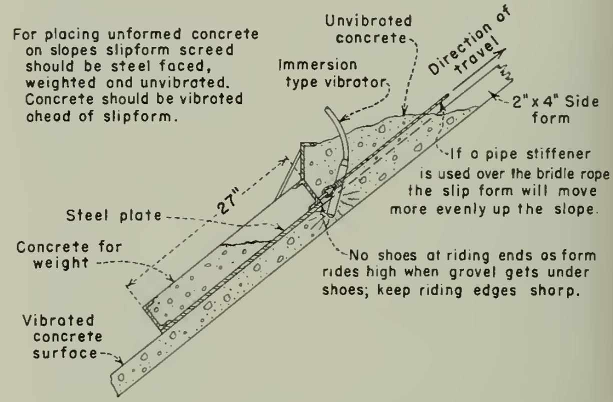


Figure F–13. **Placing unformed concrete on slopes.**

D. BIBLIOGRAPHY

F–31. *Bibliography.*

[1] "Concrete Manual," sixth edition, revised 1956, U.S. Department of the Interior, Bureau of Reclamation (for sale by the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., $2.50).

[2] ———, appendix, designation 22.

[3] "Concrete Manual," appendix, designations 9 and 10.

[4] ———, appendix, designations 4 and 5.

[5] ———, appendix, designations 23 and 24.

[6] ———, appendix, designation 12.

[7] Price, W. H., "Factors Influencing Concrete Strength," American Concrete Institute Proceedings, vol. 47, 1951, pp. 417–432.

Sample Specifications

N. F. LARKINS AND E. R. LEWANDOWSKI[1]

G–1. ***Introduction.***—Designs are based on assumptions regarding the quality of work which will be obtained during construction. It is through the means of specifications that the assumed quality is described, and it is important that conformance to the specifications be obtained for all work.

This appendix includes specifications for the various items of work and structure components whose designs are treated in this text. For the construction of a particular dam, these specifications would have to be supplemented by descriptions of items for payment, description and classification of concrete items, and by specifications for other work such as painting, installation of equipment and metalwork, etc. Not all of the specifications included herein are applicable for any one dam as they cover alternative methods and a wide variety of construction details, not all of which would be performed at any one site.

These specifications are abstracted, with slight modifications, from guide specifications normally used by the Bureau of Reclamation. The designation "contracting authority" as used in these specifications applies to the owner of the dam or his authorized representative, as appropriate.

A. EXCAVATION

G–2. ***Clearing.***—The areas to be occupied by the permanent construction required under these specifications and the surfaces of all borrow pits *(and stockpile sites) where in the judgment of the contracting authority, clearing is necessary, shall be cleared of all trees, stumps, roots, brush, rubbish, and other objectionable matter. *(Waste-pile sites in disposal areas shall be cleared of all trees and brush as directed.) Such materials from clearing operations shall become the property of the contractor and shall be burned, removed from the site of the work before the date of completion, or otherwise disposed of as approved. No trees shall be cut outside of areas mentioned above without specific approval, and all trees designated by the contracting authority shall be protected carefully from damage by the contractor's construction operations. All materials to be burned shall be piled neatly and when in suitable condition shall be burned completely. Piling for burning shall be done in such manner and in such locations as to cause the least fire risk. Burning shall be done at such times and under such regulations as *(proper Federal Forest Service officials will direct) *(may be approved by the contracting authority in accordance with the applicable laws of   ). All burning shall be so thorough that the materials are reduced to ashes. The contractor shall at all times take special precautions to prevent fire from spreading to areas beyond the limits of the cleared areas and shall have available at all times suitable equipment and supplies for use in preventing and suppressing fires.

[1] Engineer, Concrete Laboratory Branch, and Engineer, Earth Dams Section, Bureau of Reclamation.

*Revise or delete as appropriate.

The cost of clearing shall be included in the unit prices bid in the schedule for the various items of work.

G–3. *Classification of Excavation.*[2]—Except as otherwise provided in these specifications, material excavated will be measured and classified in excavation, to the lines shown on the drawings or as provided in these specifications, and will be classified for payment as follows:

(a) *Rock Excavation.*—Rock excavation includes all solid rock in place which cannot be removed until loosened by blasting, barring, or wedging and all boulders or detached pieces of solid rock more than 1 cubic yard in volume. Solid rock under this class, as distinguished from soft or disintegrated rock under common excavation, which also required blasting before removal, is defined as sound rock of such hardness and texture that it cannot be loosened or broken down by hand-drifting picks. No material, except boulders or detached pieces of solid rock, will be classified as rock excavation, which is not actually loosened by blasting before removal, unless blasting is prohibited and barring, wedging, or similar methods are prescribed by written order of the contracting authority.

(b) *Common Excavation.*—Common excavation includes all material other than rock excavation, including, but not restricted to earth, gravel, and also such hard and compact material as hardpan, cemented gravel, and soft or disintegrated rock, which cannot be removed efficiently by excavating machinery until loosened by blasting; also all boulders or detached pieces of solid rock not exceeding 1 cubic yard in volume.

No additional allowance above the unit prices bid in the schedule for excavation will be made on account of any of the material being wet or frozen.

The contracting authority's representative and contractor or the contractor's representative shall be present during classification of material excavated. On written request of the contractor, made within 10 days after the receipt of any monthly estimate, a statement of the quantities and classifications of excavation between successive stations or in otherwise designated locations included in said estimate will be furnished to the contractor within 10 days after the receipt of such request. This statement will be considered as satisfactory to the contractor unless specific objections thereto, with reasons therefor, are filed with the contracting authority, in writing, within 10 days after receipt of said statement by the contractor or the contractor's representative on the work. Failure to file such written objections with reasons therefor within said 10 days shall be considered a waiver of all claims based on alleged erroneous estimates of quantities or incorrect classification of materials for the work covered by such statement.

G–4. *Classification of Excavation (Alternate).*[3]—Materials excavated will not be classified for payment. Except as otherwise provided in these specifications, material excavated will be measured in excavation, to the lines shown on the drawings or as provided in these specifications, and all materials so required to be excavated will be paid for at the unit prices per cubic yard bid in the schedule for excavation. No additional allowance above the unit price bid in the schedule will be made on account of any of the material being wet or frozen. Bidders and the contractor must assume all responsibility for deductions and conclusions as to the nature of the materials to be excavated and the difficulties of making and maintaining the required excavations.

[4] Where the terms "rock" and "rock excavation" and "common" and "common excavation" are used in these specifications the following definitions shall apply:

(a) *Rock Excavation.*—Rock excavation includes all solid rock in place which cannot be removed until loosened by blasting, barring, wedging, and all boulders or detached pieces of solid rock more than 1 cubic yard in volume. Solid rock under this class, as distinguished from soft or disintegrated rock under common excavation, which also required blasting before removal, is defined as sound rock of such hardness and texture that it cannot be loosened or broken down by hand-drifting picks. No material, except boulders or detached pieces of solid rock, will be classified as rock excavation, which is not actually loosened by blasting before removal, unless blasting is prohibited and barring, wedging, or similar methods are prescribed by written order of the contracting authority.

---

[2] When excavations are not classified for payment use sec. G–4.

[3] When excavated materials are classified for payment use sec. G–3.

[4] Delete the remainder of sec. G–4 if the terms "rock" and "common" are not used in the specifications.

(b) *Common Excavation.* Common excavation includes all material other than rock excavation; including, but not restricted to earth, gravel, and also such hard and compact material as hardpan, cemented gravel, and soft or disintegrated rock, which cannot be removed efficiently by excavating machinery until loosened by blasting; also all boulders or detached pieces of solid rock not exceeding 1 cubic yard in volume.

G-5. *Definitions of Materials.* *(Materials excavated will not be classified for payment.) For purposes of these specifications *(other than for payment) materials of earthwork and embankment construction are defined in detail as follows:

(a) *Rock.* A solid mass of mineral material, exceeding 1 cubic yard in volume, of such hardness and texture that it cannot be broken down with a hand-drifting pick.

(b) *Common Material.* All earth materials which do not meet the requirements of rock as defined in (a) above.

(c) *Chalk (Chalk Rock).*—A material of variable hardness consisting of a consolidated aggregation of very fine particles, mainly calcium carbonate, which is usually buff in color but which may range from white to dark gray.

(d) *Shale.* A consolidated, partially laminated, fine-grained material having a tendency to split along lamination or bedding planes. It may range from a clay-like material which when cut or scraped with a knife produces a slick surface with shiny appearance, to a partially cemented material which, although it can be scratched with a knife, can be cut only with difficulty and produces a dull, fine-grained surface.

(e) *Tuff.*—A material composed of the finer kinds of volcanic detritus, usually more or less stratified, and in various states of consolidation or induration.

(f) *Soil Components.*—

(1) *Clay.*—Plastic soil which passes a United States Standard No. 200 sieve.

(2) *Silt.*—Nonplastic soil which passes a United States Standard No. 200 sieve.

(3) *Sand.* Mineral grains which pass a United States Standard No. 4 sieve and are retained on a No. 200 sieve.

(4) *Gravel.*—Pieces of rock which are not greater than 3 inches in maximum dimension, and are retained on a United States Standard No. 4 sieve.

(g) *Other Materials.*

(1) *Cobbles.*—Rounded pieces of rock which are not greater than 12 inches, but are larger than 3 inches in maximum dimension.

(2) *Boulders.* Rounded pieces of rock which are not greater than 1 cubic yard in volume, but are larger than 12 inches in maximum dimension.

(3) *Rock or tuff or chalk fragments.*—Pieces of rock or tuff or chalk which are not rounded and which are not greater than 1 cubic yard in volume.

G-6. *Open-Cut Excavation, General.*[5] (a) *General.*—All open-cut excavation shall be performed in accordance with this section to the lines, grades, and dimensions shown on the drawings or established by the contracting authority. During the progress of the work, it may be found necessary or desirable to vary the slopes or the dimensions of the excavations from those specified herein. Any increase or decrease of quantities excavated as a result of such changes will be covered in the estimates. Should it be determined by the contracting authority that unit costs will be increased or decreased as a result of such changes, an equitable adjustment of unit prices will be made.

All necessary precautions shall be taken to preserve the material below and beyond the lines of all excavation in the soundest possible condition. Any damage to the work due to the contractor's operations, including shattering of the material beyond the required excavation lines, shall be repaired at the expense of and by the contractor. Any and all excess excavation for the convenience of the contractor or overexcavation performed by the contractor for any purpose or reason, except as may be ordered in writing by the contracting authority, and whether or not due to the fault of the contractor, shall be at the expense of the contractor. Where required to complete the work, all such excess excavation and overexcavation shall be refilled with materials furnished and placed at the expense of and by the contractor: *Provided,* That payment will be made for cement used in concrete placed to refill such excess excavation or overexcavation unless such excess excavation or overexcavation is caused by careless

*Revise or delete as appropriate.

[5] Revise section as required if used for specifications for concrete dam construction

excavation or is intentionally performed by the contractor to facilitate his operations, as determined by the contracting authority. Slopes shattered or loosened by blasting shall be taken down at the expense of and by the contractor.

All excavation for embankment and structure foundations shall be performed in the dry. No excavation shall be made in frozen materials without written approval. No additional allowance above the unit prices per cubic yard bid in the schedule for excavation will be made on account of any of the materials being wet or frozen.

Where not to be covered with concrete or pervious blanket, excavations shall be made to the full dimensions required and shall be finished to the prescribed lines and grades *(except that sharp points of undisturbed ledge rock will be permitted to extend within the prescribed lines not more than 6 inches).

All shale-foundation surfaces shall be protected from freezing and air-slaking by leaving a temporary cover of ------ feet of unexcavated material. The final ------ feet of excavation above shale-foundation surfaces shall be performed by continuous operation during nonfreezing weather. Such excavation shall be followed without delay by placement of embankment material as required or by the application of protective coatings as provided in section G–15 (Protection of Finished Structure Excavations), and placement of concrete or pervious blanket material. Exposed finished excavated shale surfaces shall be kept moist at all times to prevent evaporation of the natural moisture in the material and such surfaces shall also be protected from freezing.

All chalk-foundation surfaces shall be protected from freezing by leaving a temporary cover of ------ feet of unexcavated material. The final ------ feet of excavation above chalk-foundation surfaces need not be performed by continuous operation but all such excavation shall be performed in nonfreezing weather. Exposed finished excavated chalk surfaces will not require the addition of moisture or protective coatings, but such finished surfaces shall be protected from freezing.

No allowance above the unit prices bid in the schedule will be made because of the requirements set forth herein for the protection of shale and chalk foundations from freezing or air slaking.

(b) *Structure Foundations.*—

(1) *Common material.*—The bottom and side slopes of common material upon or against which concrete is to be placed shall be finished accurately to the established lines and grades, and loose materials on surfaces so prepared shall be moistened with water and tamped or rolled with suitable tools and equipment to form a firm foundation for the concrete structure. If, at any point in common material, material is excavated beyond the established excavation lines, for any reason except by written orders from the contracting authority, the overexcavation shall be filled with selected materials, in layers not more than 6 inches thick, moistened, and thoroughly compacted by tamping or rolling. If, at any point in common material, the natural foundation material is disturbed or loosened, for any reason, it shall be consolidated by tamping or rolling, or where directed, it shall be removed and replaced with selected material, which shall be thoroughly compacted. The cost of all work required in the preparation of structure foundations shall be included in the applicable unit price bid in the schedule for excavation: *Provided,* That where the material is unsuited to form a firm foundation, further excavation and refill will be ordered in writing by the contracting authority, and, subject to the provisions of subsection (a) for an equitable adjustment of unit prices, payment therefor will be made as follows:

a. Payment for additional excavation when ordered in writing to remove unsuitable foundation materials or for excavation of excess roller compacted earthfill material, placed as described under b below, will be made at the unit price per cubic yard bid in the schedule for excavation for the structure for which the excavation is made. Payment for excavation and transportation of selected earthfill materials for use in refilling will be made at the unit price bid in the schedule for excavation of the material used for such refill.

b. In excavations where compaction with

*Revise or delete as appropriate.

the roller specified for use on the dam embankment is practical and desirable, the foundations shall be prepared, and the refill materials shall be selected, placed, moistened, and compacted as provided in subsections (b), (c), (d), (e) and (g) of section G-21 (Earthfill in Dam Embankment). The selected earthfill materials shall be placed and compacted to a depth of 18 inches above the established elevation of the structure foundation, and after compaction, such excess material shall be excavated to the established elevation of the structure foundation. Payment for placing and compacting refill materials as described above will be made at the unit price per cubic yard bid in the schedule for earthfill in dam embankment, impervious zone.

c. In excavations where compaction with the roller specified for use on the dam embankment is impractical or undesirable, the refilling and compacting of refill material shall be performed in accordance with section G-22 (Specially Compacted Earthfill). Payment for placing and compacting the refill material will be made at the unit price per cubic yard bid in the schedule for specially compacted earthfill.

Excess excavation for the convenience of the contractor or overexcavation performed by the contractor without written orders of the contracting authority shall be filled with selected material and compacted, as directed, in accordance with b and c above, except that all such work shall be at the expense of the contractor.

(2) *(*Rock, shale*) or *(*chalk material*). The bottom and side slopes of *(rock, shale) or *(chalk material) upon or against which concrete or pervious blanket material is to be placed shall be excavated to the required dimensions as shown on the drawings or established by the contracting authority. No material will be permitted to extend within the neat lines of the structure. If, at any point in *(rock, shale) or *(chalk material), upon written orders from the contracting authority, material is excavated beyond the limits required to receive the structure, the additional excavation shall be filled solidly with concrete, or with pervious blanket materials where applicable. Subject to the provisions of subsection (a) for equitable adjustment of unit prices, payment for such additional excavation will be made at the unit price per cubic yard bid in the schedule for excavation for the structure involved. Payment for concrete and/or pervious blanket material placed in such additional excavation will be made at the applicable unit price per cubic yard bid in the schedule for *(concrete in the structure involved) *(concrete in backfill), or for *(pervious blanket): *Provided*, That if determined by the contracting authority that unit costs will be increased or decreased as a result of such changes an equitable adjustment of unit prices will be made. Refill of excess excavation or overexcavation caused by careless excavation or intentionally performed by the contractor to facilitate his operations, as determined by the contracting authority, shall be in accordance with subsection (a).

(c) *Excavated Materials*.—So far as practicable, as determined by the contracting authority, all suitable materials from excavations for specified permanent construction shall be used in the permanent construction required under these specifications.

Materials shall be selected as follows:

(Insert applicable provisions.)

The contractor's blasting and other operations in excavations shall be such that the excavations will yield as much suitable material for such construction as practicable, and shall be subject to the approval of the contracting authority. Where practicable, as determined by the contracting authority, suitable materials shall be excavated separately from the materials to be wasted and the suitable materials shall be segregated by loads during the excavation operations and shall be placed in the designated final locations directly from the excavation, or shall be placed in temporary stockpiles and later placed in the designated locations as directed by the contracting authority. In excavating materials which are suitable for use in the dam embankment, the contracting authority will designate the depths of cut which will result in the best gradation of materials, and the cuts shall be made to such designated depths.

*Revise or delete as appropriate

Excavated materials which, after drainage, are suitable for the impervious rolled earthfill portion of the dam embankment but which, when excavated, are too wet for immediate compaction in the embankment shall be placed temporarily in stockpiles until the moisture content is reduced sufficiently to permit them to be placed in the embankment, or they shall be placed on the embankment subject to the provisions of subsection (g) of section G–21 (Earthfill in Dam Embankment), relative to materials in which the moisture content is greater than that required for proper compaction. Should cobbles, boulders, or rock fragments having maximum dimensions of more than 5 inches be found in otherwise approved earthfill materials, they shall be removed by the contractor either at the site of the excavation or after being transported to the earthfill but before the materials are rolled and compacted. Such rock materials shall be placed in other portions of the dam embankment or wasted, as directed.

Excavated materials which are unsuitable for or are in excess of dam embankment or other earthwork requirements, as determined by the contracting authority, shall be wasted as provided in section G–19 (Disposal of Excavated Material).

(d) *Measurement and Payment.*—Excavated material will be measured, for payment, in excavation to the lines shown on the drawings or described in these specifications, and will include only material that is actually removed at the direction of the contracting authority: *Provided*, That where excavation lines are not shown on the drawings, the excavation will be measured to the most practicable lines, grades, and dimensions as prescribed by the contracting authority.

Where concrete or pervious blanket material is to be placed directly upon or against the excavations and the character of the material cut into is such that the material can be trimmed efficiently to accurate dimensions by ordinary excavation finishing methods to the required lines of the concrete structure or overlying blanket material, as determined by the contracting authority, measurement for payment will be made only of the excavation within the neat lines of the concrete structure or established lines of pervious blanket: *Provided*, That where minimum dimensions for such concrete or pervious blanket are shown on the drawings, measurement for payment will be made to the minimum dimensions shown on the drawings or established by the contracting authority: *Provided further*, That where no dimensions are indicated on the drawings, minimum dimensions will be established by the contracting authority, and measurement, for payment, of excavation will be made to such minimum dimensions.

Where concrete or pervious blanket material is to be placed directly upon or against the excavations and the character of the material cut into is such that the material cannot be trimmed efficiently to accurate dimensions by ordinary excavation finishing methods, as determined by the contracting authority, such excavations shall be sufficient at all points to provide for the minimum dimensions of concrete or pervious blanket shown on the drawings, and the prescribed average dimension shall be exceeded as little as possible, and measurement for payment thereof will be made to the prescribed average dimensions: *Provided*, That where a single dimension is given or if the elevation of the foundation is indicated on the drawings, such dimensions shall be considered as the minimum dimension and such elevation shall be considered as the elevation determining the minimum dimension, and the prescribed average dimension lines shall be considered as 3 inches outside the minimum dimension lines for the purposes of measurement, for payment, of excavation: *Provided further*, That where no dimensions or elevations are indicated on the drawings, minimum dimensions will be established by the contracting authority, and measurement, for payment, of excavation will be made to the average dimension lines 3 inches outside the minimum dimension lines as established by the contracting authority. In areas where corrugations are required under structure foundations, measurement, for payment, of excavation for the structure will be made to the average dimension lines 3 inches below the minimum dimension lines of the concrete structure as shown on the drawings or established by the contracting authority. Measurement, for payment, of excavation for sewer-pipe drains, including excavation for bedding and concrete pads, will be made to the neat lines shown on the drawings or established by the contracting authority.

Except as otherwise provided in section G–63 for Diversion and Care of River During Construction and Removal of Water from Foundations, the

unit prices bid in the schedule for excavation in open cut shall include the cost of all labor, equipment, and materials for cofferdams and other temporary construction and of all pumping, bailing, draining, and all other work necessary to maintain the excavations in good order during construction and of removing such temporary construction where required.

The unit prices bid in the schedule for excavation in open cut shall also include the entire cost of:

(1) Transportation of materials from the excavation to points of final use, to disposal areas, to temporary stockpiles, and from temporary stockpiles to points of final use.

(2) Rehandling excavated materials which have been deposited temporarily in stockpiles.

(3) Removal of oversize materials from otherwise suitable materials and disposal of the same.

(4) Disposal of excavated waste materials.

All excavated materials actually placed in completed earthwork and embankment construction will again be included for payment under appropriate items of the schedule covering such construction. No payment will be made for excavation performed in previously placed embankment, refill or backfill, except as provided in subsection (b) above.

G–7. ***Excavation for Grout Cap.***—Excavation for grout cap shall be performed by the use of hand tools and approved mechanical equipment, in such a manner as to prevent shattering of the sides and bottom of the excavation. At the option of the contractor and with the approval of the contracting authority, line drilling and light blasting in blasting holes or other methods may be employed. If line drilling and light blasting are employed the diameter, spacing, and depth of the line drilling holes and the blasting holes shall be subject to the approval of the contracting authority, and the spacing shall be such as to insure that the material will break along the desired lines. The blasting shall be limited to approved methods which provide for successive fracturing of the worked face as the work is advanced by use of power tools and handwork. Blasting in line drill holes will not be permitted. Whenever, in the opinion of the contracting authority, further blasting might injure the surfaces upon or against which concrete is to be placed, the use of explosives shall be discontinued.

When an excavation for grout cap crosses a fault or seam the excavation shall be carried to depths shown on the drawings or as may be directed and shall be keyed into the formation on the sides of the fault or seam as directed: *Provided*, That if excavation is required to a greater depth than 8 feet measured normal to the finished surface of excavation for dam embankment foundation, such excavation will be ordered in writing. The contractor shall furnish all materials to support the sides of the excavation where necessary, and all supports shall be removed before or during the placing of concrete. At the option of the contractor, excavating for grout cap may be performed during nonfreezing weather in advance of final excavation to finished surfaces of dam embankment foundation. Such excavation, however, shall be followed without delay by placement of concrete in grout cap.

Measurement, for payment, of excavation for grout cap will be made to the prescribed average dimension in width, and to the designated depth measured normal to the finished surfaces of excavation, *(and will not include the ___ -foot-thick temporary cover of unexcavated material).

Payment for excavation for grout cap will be made at the applicable unit price per cubic yard bid in the schedule for excavation for grout cap up to 5 feet in depth and for excavation for grout cap between depths of 5 feet and 8 feet, which unit price shall include the entire cost of all work described in this section and the cost of furnishing, installing, and removing supports. The requirement for excavation for grout cap between depths of 5 feet and 8 feet will be determined by the contracting authority, and the contractor will be entitled to no additional allowance above the unit price bid therefor in the schedule by reason of any amount or none or the work for this item being required.

G–8. ***Drilling Line Holes for Rock Excavation.***—Rock excavation, where directed by the contracting authority, shall be formed by line drilling and broaching. The diameter of the holes for line drilling shall be subject to approval. The spacing of the holes shall be as directed by the contracting authority, and shall be sufficiently close to insure that the rock will break along

*Revise or delete as appropriate.

the desired lines. No blasting will be permitted in the holes along the sides of the excavation, but light blasting will be permitted in the areas adjacent to lines of holes: *Provided*, That whenever, in the opinion of the contracting authority, further blasting might injure the rock upon or against which concrete is to be placed, the use of explosives shall be discontinued and the excavation shall be completed by wedging, barring, or other suitable methods.

Payment for drilling line holes along the sides of the excavation will be made at the unit price per linear foot bid in the schedule for drilling line holes for rock excavation, and only the length of holes actually drilled into the rock along the sides of the excavation at the direction of the contracting authority will be considered in making measurements for payment. No payment will be made to the contractor for drilling holes for blasting purposes.

G–9. *Excavation for Dam Embankment Foundation (Earthfill Dam).*—The item of the schedule for excavation for dam embankment foundation includes all excavation for the foundation of the dam embankment, including stripping.

The entire area to be occupied by the dam embankment or such portions thereof as may be directed shall be stripped to a sufficient depth to remove all materials not suitable, as determined by the contracting authority, for the foundation of the dam embankment. The unsuitable materials to be removed include all topsoil, rubbish, vegetable matter including stumps and roots, and all other perishable and objectionable material.

A cutoff trench shall be excavated in the dam embankment foundation to the established lines and grades and all loose, soft, and disintegrated material shall be removed to the extent directed. Removal of boulders may be required. The contractor will not be required to excavate the cutoff trench to depths below elevation ______ at the unit price per cubic yard bid in the schedule for excavation for dam embankment foundation.

The contemplated alinements and cross-sectional dimensions shown will be subject to such changes as may be found necessary by the contracting authority to adopt the foundations to the conditions disclosed by the excavations. Accurate trimming of the slopes of the excavations will not be required, but the excavations shall conform as closely as practicable to the established lines and grades.

Payment for excavation for dam embankment foundation will be made at the unit price per cubic yard bid therefor in the schedule.

G–10. *Excavation in Open Cut for Concrete Structures.*—The item of the schedule for excavation in open cut for concrete structures includes all open-cut excavation, including stripping, as shown on the drawings, for the concrete structures as follows:

(Here usually are listed descriptions and locations of the excavations involved under this excavation pay item, together with the dividing limits for measurement for payment if these excavations are contiguous with other excavations for which separate payment is made. This section is also used for specifications for construction of a concrete dam.)

Payment for excavation in open cut for concrete structures will be made at the unit price per cubic yard bid therefor in the schedule.

G–11. *Excavation in Tunnel and Shaft.*[6]—Excavations shall be made to the lines, grades, and dimensions shown on the drawings or established by the contracting authority. The general dimensions, arrangements, and details of typical sections of the tunnel and shaft are shown on the drawings. *(All final excavated surfaces of the tunnel and shaft shall, within 1 hour after exposure, be protected by a coating one-sixteenth inch in minimum thickness of emulsified asphalt, RS–1. Emulsified asphalt shall be in accordance with Federal Specifications SS–A–674b, shall be applied by approved methods, and so far as practicable as determined by the contracting authority, shall be applied before placement of permanent tunnel and shaft supports or temporary timbering.) Permanent tunnel and shaft supports shall be furnished and installed by the contractor where necessary, as determined by the contracting authority, in accordance with section G–12 (Permanent Tunnel and Shaft Supports). Temporary timbering may be used in accordance with section G–13 (Temporary Timbering in Tunnel and Shaft). During construction the tunnel and shaft shall be drained, lighted, and ventilated in accordance with section G–14 (Draining, Lighting, and Ventilating Tunnel and Shaft).

[6] Secs. G–11, G–12, G–13, and G–14 are applicable to both tunnels and shafts. The wording should be revised depending on whether one or more tunnels and one or more shafts are involved.

*Revise or delete as appropriate.

The "A" lines shown on the typical sections of the drawings are lines within which no unexcavated material of any kind and no supports, other than permanent structural-steel supports, will be permitted to remain. The "B" lines shown on the typical sections are the outside limits to which measurement, for payment, of excavation will be made, and measurement for payment will in all cases be made to the "B" lines regardless of whether the limits of the actual excavation fall inside or outside of the "B" lines.

The nature of the materials being excavated may make it necessary, as determined by the contracting authority, to increase the distance between the "A" line and the finished interior lining surfaces of the tunnel and shaft, in which event the position of the "B" line will be changed in such a way as to maintain at every point the same distance between the "A" and "B" lines as existed before the position of the "A" line was moved: *Provided*, That where steel supports, other than steel liner plates alone, are used, the "B" line will be moved so as to be located outside of the outer face of the approved steel supports the distance shown on the drawings. The contractor shall be entitled to no additional compensation because of such changes other than that resulting from the increased quantities due to the new positions of the "A" and "B" lines: *Provided*, That any additional excavation required on account of enlargement of section, ordered in writing after the completion of the excavation of the section to the previously described dimensions, will be paid for as extra work. Where foundation conditions, as determined by the contracting authority, require additional excavation for extending structural-steel ribs, *(wallplates, foot beams, footplates), or other approved structural-steel members, such excavation will be included for payment under the items of the schedule for excavation in tunnel and shaft.

The contractor shall use every precaution to avoid loosening material beyond the "B" lines. All drilling and blasting shall be performed carefully so that the material outside the "B" lines will not be shattered. Any damage to or displacement of tunnel supports and any damage to any other part of the work caused by blasting or any other operations of the contractor shall be repaired at the expense of and by the contractor and in an approved manner.

Immediately following excavation in unsupported sections, all loosened material either inside or outside of the "B" lines that, in the opinion of the contracting authority, is likely to fall shall be removed.

All material projecting inside the "A" lines shall be removed by the contractor as part of the work described in this section. The removal of such projections within the "A" lines may be performed at any time during the progress of the work: *Provided*, That immediately before the concrete lining is placed, the contractor will be required to remove all material then extending within the "A" lines. Excavated materials shall be placed in *(dam embankment or) ________.

Measurement, for payment, of excavation in tunnel and shaft will be limited to the specified sectional dimensions and will be made along the located centerlines of the tunnel and shaft only for such reaches of the tunnel and shaft as are excavated by tunneling and shaft driving methods. Payment for excavation in tunnel and shaft will be made at the applicable unit prices per cubic yard bid therefor in the schedule, which unit price shall include the entire cost of excavation, transportation, and disposal of excavated materials *(and furnishing and applying RS-1 coating to final excavated surfaces of the tunnel and shaft).

G-12. *Permanent Tunnel and Shaft Supports.*—(a) *General.*—Suitable permanent structural-steel supports, as provided in subsection (b) below, shall be used to support the roof and sides of the tunnel and shaft, where required, and as approved by the contracting authority. *(Tunnel roof support bolts used as roof supports or side anchors, and chain-link woven wire fabric, shall be installed in the tunnel, gate chamber, and shaft where required as provided in subsections (c) and (d) below.) The space between the permanent structural-steel liner plates and/or between continuous steel lagging and the excavated surfaces shall be filled with coarse aggregate or rock fragments not less than five-eighths inch in maximum dimension and grouted.

(b) *Permanent Structural-Steel Supports.*—The contractor shall furnish all permanent structural-steel supports consisting of steel ribs, *(wallplates,

* Revise or delete as appropriate.

foot beams, footplates), lagging, liner plates, and other approved structural-steel members, complete with bolts, nuts, wedges, tie rods, and other accessories required for assembling the permanent structural-steel supports and supporting them in place, and all temporary timber spreaders. The permanent steel supports shall be placed in an approved manner to the established lines, grades, and dimensions, and shall be maintained by the contractor in proper condition and alinement until the concrete lining has been placed. The distance between structural-steel rib supports shall be as approved. Temporary timber spreaders shall be furnished and installed by the contractor where required and shall be removed before placement of concrete lining. No direct payment will be made to the contractor for temporary timber spreaders.

Typical approved permanent supports for the tunnel consisting of structural-steel ribs, with or without steel lagging or liner plates; or steel tunnel-liner plates without structural-steel ribs, and other approved structural-steel members are shown on drawing No. ______. Other permanent structural-steel supports may be used as approved. If permanent supports are required for irregular sections such as portals, transitions, and tunnel enlargements, such supports shall be in accordance with designs approved by the contracting authority. Approved types of permanent steel supports are shown on the drawings, but if these types are found to be inadequate they may be modified from time to time, subject to approval. The details of the permanent structural-steel supports including size, weights, and materials, and the installation in all parts of the tunnel and shaft shall be subject to approval. The amount of permanent supports that will be required is uncertain, and the contractor shall be entitled to no additional allowance above the unit prices bid in the schedule by reason of a larger or smaller amount or no permanent supports being required, except that differences of actual quantities from the schedule quantities will be covered in the estimates. Any required repair to or replacement of supports due to the contractor's operations shall be made at the expense of and by the contractor.

Nothing contained in this section shall prevent the contractor, at his own expense, from erecting such amounts of temporary supports as he may consider necessary, or from using heavier permanent structural-steel supports than approved, if use of such heavier members results in no increased cost to the contracting authority, and no statement herein shall be construed to relieve the contractor from sole responsibility for the safety of the tunnel and shaft or for liability for injuries to or deaths of persons or damage to property.

Measurement, for payment, of permanent structural-steel supports will include only the weights of the steel ribs, *(wallplates, foot beams, footplates), liner plates, lagging, and other approved structural-steel members as are approved for placing and which, in the judgment of the contracting authority, are necessary for adequate construction. Payment for furnishing and placing permanent structural-steel supports will be made at the unit price per pound bid therefor in the schedule, which unit price shall include the cost of furnishing, placing, and removing timber spreaders.

(c) *Tunnel Roof Support Bolts.*—The contractor shall furnish and install tunnel roof support bolts for use as roof supports or side anchors in the tunnel, gate chamber, and shaft, where approved or directed. The requirement for furnishing and installing tunnel roof support bolts at any location and the amount thereof shall be subject to the approval of the contracting authority. The contractor shall drill holes for the roof support bolts into the roof or sides of the tunnel, gate chamber, and shaft at locations and to depths as directed or approved by the contracting authority and shall provide and install roof support bolts complete with all required accessories including bearing plates, wedges, expansion anchors, nuts and washers. The bearing plates shall have a bearing area of not less than 36 square inches per bolt and may be steel plate, rolled-steel channel, beam or angle to which one or more roof support bolts are connected as approved by the contracting authority. The contractor may furnish roof support bolts with wedge anchors or with expansion anchors as hereinafter described. Other types of anchoring devices may be furnished if approved.

(1) Roof support bolts with wedge anchors shall be 1-inch-diameter steel bolts, slotted at one end and threaded 8 inches at the other

*Revise or delete as appropriate.

end. Bolts shall conform to the following specifications:

Minimum yield, strength, p.s.i.	Minimum tensile strength, p.s.i.	Minimum elongation (8-inch gage length)
30,000	60,000	17%

Each bolt shall be furnished with one steel wedge and one nut. The slots shall be approximately ⅛-inch wide and 6 inches long. The steel wedges shall be approximately 5½ inches long and shall be tapered to ¾-inch by ¾-inch at the blunt end. The holes for the roof support bolts with wedge anchors shall be 1¼ inches to 1½ inches in diameter, and shall be drilled accurately to the depth directed. The bolts with wedges inserted in the slots shall be placed in the drill holes and expanded by driving against the rock at the end of the drilled hole. The bolt shall be driven into the hole until the slotted portion of the bolt has expanded sufficiently to provide adequate anchorage of the bolt against the sides of the drilled hole. After the bolts are secured in the holes, bearing plates shall be placed on the bolts and a nut placed on each bolt and drawn up tight against the bearing plates so that the bearing plates have a firm bearing against the rock surface.

(2) Roof supports with expansion anchors shall be ¾-inch diameter with a fixed square head on one end and threaded 8 inches at the other end, and each bolt shall be furnished with one expansion shell anchor. Bolts shall conform to the following specifications:

Minimum yield, strength, p.s.i.	Minimum tensile strength, p.s.i.	Minimum elongation (8-inch gage length)
40,000	80,000	12%

The expansion shell anchor shall be suitable for installation in 1⅜-inch-diameter holes and shall be approved by the contracting authority. The holes for the roof support bolts with expansion anchors shall be 1⅜-inch diameter and shall be drilled to a depth not less than that directed by the contracting authority. The bolts shall be inserted through the bearing plates, expansion anchors loosely attached to the bolts, and the bolts and expansion anchors inserted into the holes until the bearing plate is snug against the rock. The bolts shall then be tightened until the required torque is developed in the bolt.

Measurement, for payment, of tunnel roof support bolts will be made of the length of hole drilled for the installation of the bolts at the direction of the contracting authority: *Provided*, That any bolt furnished and installed in a hole drilled less than 6 feet deep at the direction of the contracting authority will be paid for as a bolt installed in a hole 6 feet deep. Payment for furnishing and installing tunnel roof support bolts will be made at the unit price per linear foot bid therefor in the schedule, which unit price shall include the cost of drilling the holes and furnishing and installing the bolts, expansion devices, and nuts. Payment for furnishing and installing bearing plates will be made at the unit price per pound bid in the schedule for furnishing and placing permanent structural-steel supports.

(d) *Chain-Link Woven Wire Fabric for Tunnel Roof Support Bolts*.—The contractor shall furnish and install chain-link woven wire fabric for roof support bolts or side anchors in the tunnels, gate chamber, and shafts where approved or directed. The requirement for furnishing and installing chain-link fabric at any location and the amount thereof shall be subject to the approval of the contracting authority.

The chain-link woven wire steel fabric shall conform to Federal Specification RR–F–191a, 2-inch mesh, No. 9 gage (0.148-inch) coated or uncoated. The fabric shall be placed over the previously installed roof bolts and drawn up tight against the rock surface by means of the nut and bearing plate. The contractor shall lap sections of fabric a minimum of 4 inches where practicable: *Provided*, That at connections where it is impracticable to maintain 4-inch laps as determined by the contracting authority, the contractor will be permitted to extend laps in lieu of cutting along regular lines. Final layout of the fabric and extent of lapping shall be subject to approval.

Measurement, for payment, of chain-link fabric will be made of the number of pounds of fabric actually installed in the completed work as directed and will be based on furnishing a 2-inch mesh, No. 9 gage (0.148-inch) fabric. Payment for furnishing and installing chain-link woven-wire fabric for tunnel roof support bolts will be made at the unit price per pound bid therefor in the schedule.

G–13. *Temporary Timbering in Tunnel and Shaft.*—Suitable temporary timbering, including lagging, may be used where such temporary timbering is necessary, to support the roof and sides of the tunnel and shaft. Such temporary timbering shall

be removed by the contractor before the concrete lining is placed. Lumber for temporary timbering, if used, shall be furnished by the contractor. No direct payment will be made for furnishing, erecting, and removing temporary timbering. Nothing contained in this section shall prevent the contractor, at his own expense, from erecting such amounts of temporary timbering as he may consider necessary, subject to the above provisions, nor shall it be construed to relieve the contractor from sole responsibility for the safety of the tunnel and shaft or from liability for injuries to or deaths of persons or damage to property.

The entire cost of furnishing, erecting, and removing temporary timbering shall be included in the unit prices bid in the schedule for the items of work covering the construction of tunnel and shaft.

G–14. ***Draining, Lighting, and Ventilating Tunnel and Shaft.***—The contractor shall drain the tunnel and shaft of water, as necessary, to obtain satisfactory working conditions. Pumping will be required where gravity flow to an outlet cannot be secured. The contractor shall also properly light and ventilate the tunnel and shaft during all construction operations.

The cost of all work described in this section shall be included in the unit prices bid in the schedule for the items of work covering construction of the tunnel and shaft.

G–15. ***Protection of Finished Structure Excavations.***—(a) *General.*—The contractor shall place a protective coating of sprayed material or concrete on finished excavated foundation surfaces for spillway and outlet works *(and other structures) as shown on the drawings or as directed. The type of coating to be applied to the surfaces and the locations requiring protection shall be, in general, as shown on the drawings and as hereinafter specified, and such coatings shall be applied only to portions of the areas designated to receive protective coating where in the judgment of the contracting authority they are deemed necessary.

The contractor shall not perform excavation within the final ______ feet of finished surfaces in areas which will require protective coatings until all equipment and facilities required for immediate application of protective coatings are available and in working condition. The final ______ feet of excavation in areas requiring protection shall be performed by continuous operation, and the application of protective coatings shall follow without delay. The final 12 inches of excavation in such areas shall be completed within 8 hours, and protective coating shall be applied to those areas so that finished excavated surfaces will be exposed for the shortest possible period of time which shall not exceed one-half hour. Exposed finished excavated shale surfaces shall be kept moist at all times to prevent evaporation of the natural moisture in the material and shall be protected from freezing.

Surfaces to be coated shall be cleaned of all loose material, dirt, dust, mud, standing water, and other foreign matter. Coatings of protective material shall not be applied when the air temperature is below 35° F., nor during other adverse weather as determined by the contracting authority. Temporary protective coverings, at the contractor's expense and as approved by the contracting authority, will be permitted, if necessary, until the permanent protective coatings are applied. Protective coatings damaged by frost, heat, traffic, or other causes shall be removed and replaced or repaired at the expense of and by the contractor as directed, including the cost of removal of any frost or other damaged foundation material below such protective coatings and replacement with compacted earth material or concrete as applicable. Protective coatings shall be covered with structural concrete or filter blanket material as soon as practicable, and in no event shall the protective coatings be left exposed for a period exceeding 30 days.

(b) *Sprayed Protective Coating.*—Sprayed protective coatings shall be applied to the following finished excavated surfaces: A sprayed protective coating of bituminous material shall be applied to finished excavated surfaces by methods approved by the contracting authority. Material for sprayed protective coating shall be furnished by the contractor and shall be asphalt cutback, Designation RC–1, in accordance with Federal Specification SS–A–671a. The coating shall be applied to a thickness of one-sixteenth inch in such a manner that a uniform coating will be produced which will protect the finished excavated surface from the air.

If found necessary to protect sprayed protective coatings temporarily, the contractor at his own expense will be permitted to dust the coating with dry cement or protect the coating by

*Revise or delete as appropriate.

other approved means: *Provided*, That all such protection is removed where practicable, as approved by the contracting authority, before the overlying material is placed.

(c) *Concrete Protective Coating.*—A protective coating of concrete shall be applied to the following finished excavated shale surfaces: Concrete shall be applied so that the contact surface is covered to a depth of 2 inches above the minimum excavation lines shown on the drawings or established by the contracting authority. The concrete shall conform to the applicable provisions specified in part D of this appendix. The exposed surfaces of concrete protective coating shall be considered as a construction joint and as such shall receive the same treatment required by the provisions of section G-57 (Preparation for Placing) prior to placement of structural concrete on the concrete protective coating. Either water or membrane curing may be used on concrete protective coating, but if membrane curing is used, sealing compound shall be removed by approved methods at the contractor's expense before placement of structural concrete. The method of placing the concrete shall be subject to the approval of the contracting authority. Concrete for protective coating shall contain ¾-inch maximum size aggregate.

(d) *Measurement and Payment.*—Measurement, for payment, of sprayed protective coating and of concrete protective coating will be made of the number of square yards of area actually coated at the direction of the contracting authority. Payment for sprayed protective coating will be made at the unit price per square yard bid therefor in the schedule, which unit price shall include the cost of all work, equipment and materials required for application of sprayed protective coating. No payment will be made for cement used to dust sprayed protective coating. Payment for concrete protective coating will be made at the unit price per square yard bid therefor in the schedule, which unit price shall cover only the cost of the additional work required for placing concrete protective coating, since concrete placed for protective coating will be included for payment in the item of the schedule for concrete in the structure involved. Payment for cement used in concrete protective coating will be made at the unit price per barrel bid in the schedule for furnishing and handling cement.

G-16. *Borrow Areas.* (a) *General.*—All materials required for:

(1) Construction of dam embankment zones ____, ____, and ____.

(2) Pervious backfill.

(3) ____.

*(which are not available from excavations required for permanent construction under these specifications), shall be obtained from borrow areas ____, ____, ____, and ____ shown on drawing No. ____. The location and extent of all borrow pits within borrow areas shall be as directed. The contracting authority reserves the right to change the limits or location of borrow pits within the limits of the borrow areas in order to obtain suitable material and to minimize stripping operations.

The contracting authority will designate the depths of cut in all parts of the borrow pits, and the cuts shall be made to such designated depths. The earthfill material delivered on the dam embankment shall be equivalent to a mixture of materials obtained from an approximately uniform cutting from the full height of the designated face of the borrow-pit excavation. *(Shallow cuts will be permitted in borrow areas ____ and ____ if unstratified materials with uniform water content are encountered.) The type of equipment used and the contractor's operations in the excavation of materials in borrow pits shall be such as will produce the required uniformity of mixture of the materials at the borrow pits.

The contractor shall be entitled to no additional allowance above the unit prices bid in the schedule on account of any changes ordered by the contracting authority in the amounts of materials to be secured from any borrow area, or on account of the designation by the contracting authority of the various portions of the borrow areas from which materials are to be obtained, or on account of the depths of cut which are required to be made.

Should any borrow pit be opened near the dam embankment and below the elevation of normal water surface in the reservoir, berms not less than ____ feet wide shall be left between the toe of the dam embankment and the edge of the borrow pit, with a slope of 4:1 to the bottom of the borrow pit. Excavated surfaces of borrow pits above the normal water surface shall be graded

*Revise or delete as appropriate.

to slopes not steeper than ______ and other excavated borrow-pit surfaces shall be graded to slopes not steeper than ______. Borrow pits shall be operated so as not to impair the usefulness or mar the appearance of any part of the work or any other property of the contracting authority. The surfaces of wasted material shall be left in a reasonably smooth and even condition.

(b) *Embankment Materials.*—Materials for earthfill, zone ______ and __________________ portions of the dam and __________________ embankments, shall be obtained from excavation in open cut for permanent construction required under these specifications and from borrow areas ______, ______, and ______.

Materials for sand and gravel fill, zone ______ and __________________ portions of the dam and __________________ embankments, shall be obtained from excavation in open cut for permanent construction required under these specifications and from borrow areas ______, ______, and ______.

Materials for sand, gravel, and cobble-fill, zone ______ and __________________ portions of the dam and __________________ embankments, shall be obtained from excavation in open cut for permanent construction required under these specifications and from borrow areas ______, ______, and ______.

Materials for bedding for riprap may be obtained from excavation in open cut for permanent construction required under these specifications and from borrow areas ______, ______, and ______.

Materials for selected surfacing may be obtained from borrow areas ______, ______, and ______.

(c) *Moisture and Drainage.*—The water content of the earthfill material prior to and during compaction shall be in accordance with subsection (d) of section G–21 (Earthfill in Dam Embankment). As far as practicable, the material shall be conditioned in the borrow pits before excavation. If required, moisture shall be introduced into the borrow pits for the earthfill material by irrigation, at least ______ days in advance of excavation operations, *(or at the option of the contractor, moisture may be added at the separation plant). When moisture is introduced into the borrow pits for earthfill material prior to excavation, care should be exercised to moisten the material uniformly, avoiding both excessive runoff and accumulation of water in depressions. If at any location in the borrow pits for earthfill material, before or during excavation operations, there is excessive moisture, as determined by the contracting authority, steps shall be taken to reduce the moisture by selective excavation to secure the materials whose moisture content is closest to optimum; by excavating drainage ditches; by allowing adequate time for curing or drying; or by any other approved means. Borrow pits for sand and gravel and cobble-fill material will not require preconditioning by irrigation but may require draining. The contractor, at his option, may excavate sand and gravel and cobble-fill material under water: *Provided,* That the contractor shall be entitled to no additional allowance above the unit prices bid in the schedule on account of delays due to poor trafficability in the borrow area or on the embankment or any other difficulties due to overly wet conditions resulting from such operations.

To avoid the formation of pools in borrow pits during the excavation operations or in borrow pits above elevation ______ after the excavation operations are completed, drainage ditches from borrow pits to the nearest outlets shall be excavated by the contractor where, in the opinion of the contracting authority, such drainage ditches are necessary.

No direct payment will be made for irrigation of borrow areas, *(for addition of moisture at separation plant), for excavating drainage ditches, or for any other operations necessary to properly condition the material; and the entire cost of such irrigation *(addition of moisture), excavation, or other operations, shall be included in the unit price per cubic yard bid in the schedule for excavation in borrow areas.

(d) *Stripping and Waste.*—Borrow-pit sites shall be cleared as provided in section G–2 (Clearing). Borrow pits will be designated by the contracting authority as the work progresses, and stripping operations shall be limited only to designated borrow pits. The contractor shall carefully strip the sites of designated borrow pits of topsoil, sod, loam, and other matter which is unsuited for the purposes for which the borrow pit is to be excavated. The contractor shall maintain the stripped surfaces of borrow pits free of vegetation

*Revise or delete as appropriate.

until excavation operations in that borrow pit are completed, and the contractor shall be entitled to no additional allowance above the unit prices bid in the schedule because of this requirement. *(Materials from stripping which are suitable for topsoil for seeding shall be selected during stripping operations and stockpiled adjacent to borrow areas for future use.) Materials from stripping *(which are not suited for topsoil for seeding) shall be disposed of in exhausted borrow pits, or in approved areas adjacent to borrow pits, or as provided in section G-19 (Disposal of Excavated Material).

Measurement, for payment, of stripping borrow pits will be made in excavation and will include only the stripping in locations and to the depths as directed by the contracting authority. Payment for stripping borrow pits and disposal of materials from such stripping will be made at the unit price per cubic yard bid in the schedule for excavation, stripping borrow pits, *(which unit price shall include the cost of selecting and placing topsoil in stockpiles in locations adjacent to the borrow areas, as directed). If materials unsuitable or not required for permanent construction purposes are found in any borrow pit, such materials shall be left in place or excavated and wasted, as directed, and payment for such excavation and disposal of unsuitable or excess materials will be made at the unit price per cubic yard bid in the schedule for excavation, stripping borrow pits.

(e) *Separation*[7].—Materials from borrow areas ------ shall be separated before placement in dam embankment, zone ------ or ------ and --------------------. The contractor shall construct separation plant facilities which will separate cobbles, boulders, and rock fragments having maximum dimensions greater than------ inches from all other material. Material ------ inches or less in maximum dimensions shall be placed in dam embankment zone ------ or ------, and cobbles, boulders, and rock fragments having maximum dimensions greater than ------ shall be placed in--------------------: *Provided*, That cobbles, boulders, and rock fragments larger than ------ inches in maximum dimensions shall be placed in the outer slopes of ------ -------------- or shall be embedded in that zone, so as not to interfere with the compaction operations.

The cost of separation as described in this subsection shall be included in the applicable unit price per cubic yard bid in the schedule for excavation in borrow areas ------ and ------, separation, and transportation to dam embankment.

(f) *Measurement and Payment.*—Measurement, for payment, of excavation in borrow areas will be made in excavation only and to the excavation lines prescribed by the contracting authority. Payment for excavation in borrow areas and transportation to dam embankment, *(and for excavation in borrow areas, separation, and transportation to dam embankment), will be made at the applicable unit prices per cubic yard bid therefor in the schedule. Except as provided in subsection (d) above, the above unit prices shall include the entire cost of irrigation, *(addition of moisture at separation plant), drainage, and of all other operations required by this section. All materials from borrow pits placed in dam embankment, zones------, in--------------------, and in backfill will again be included for payment under the applicable items of the schedule for placing such earthwork.

If the contractor elects to obtain concrete aggregates, gravel for drains,--------------------, or other materials for which the cost of furnishing or procuring is included in other items of work, no payment will be made for stripping or excavation of such materials obtained from borrow areas. The contractor shall keep his operations for the production of these materials separate and distinct from his other borrow area operations.

G-17. *Rock Deposits (Rock to be Furnished by Contractor).*—Rock fragments of the quality and gradations specified herein shall be furnished by the contractor for use in bedding for riprap and riprap to be placed and stockpiled and for other permanent construction required under these specifications.

(a) *Quality.*—The rock fragments shall meet the following requirements as to quality:

(1) Individual rock fragments shall be dense, sound, and resistant to abrasion and shall be free of cracks, seams, and other defects that would tend to increase unduly their destruction by water and frost actions.

[7] This section will be applicable only for materials requiring separation.

*Revise or delete as appropriate.

(2) Samples prepared in accordance with applicable designations of the Bureau of Reclamation Concrete Manual, sixth edition, shall meet the following requirements when tested by the procedures described in the respective designations.

Test	Designation	Requirements
Specific gravity (saturated surface-dry basis).	10	Greater than.
Soundness (sodium sulfate method).	19	Less than 10 percent loss of weight after 5 cycles.
Abrasion (using Los Angeles machine grading A).	21	Less than 35 percent loss of weight after 500 revolutions.

Samples of ______ and ______ [8] from the following locations have been tested and found suitable: (List locations.)

Bidders and the contractor are cautioned that the above-mentioned deposits may be variable in quality and the sizes and quantity of rock fragments that may be obtained from any source is unknown. The contractor will be responsible for furnishing suitable rock fragments, for making necessary arrangements with property owners for right-of-way, and for payment of required royalties.

(b) *Gradations.*—Rock fragments shall be reasonably well graded to size or volume to meet requirements as follows:

(1) Riprap materials shall be graded as follows:

Nominal thickness, inches	Gradation, percentage of stones of various weights (pounds)			
	Maximum size	25 percent greater than	45 to 75 percent from — to —	25 percent less than [1]
Note.—Gradation should be determined in accordance with table 17 (ch. V)				

[1] Sand and rock dust to be less than 5 percent.

(2) Bedding for riprap material shall consist of rock fragments reasonably well graded from $\frac{3}{16}$ inch to $3\frac{1}{2}$ inches in size.

(c) *Sampling and Testing.*—The contractor shall furnish, to the contracting authority at the damsite, without cost, such samples of rock fragments for testing as may be required by the contracting authority from proposed quarry sites and from rock fragments delivered to the damsite. The contracting authority reserves the right to make inspections of quarry sites and quarries. The approval of some rock fragments from a particular quarry site shall not be construed as constituting the approval of all rock fragments taken from that quarry, and the contractor will be held responsible for the specified quality and gradation of rock fragments delivered to the damsite. All rock fragments not meeting the requirements of these specifications, as determined by tests and/or inspection at the quarries and damsite, will be rejected.

G–18. *Rock Source (Source Furnished by Contracting Authority).*—All rock materials required for construction of:

(1) ________________________, and

(2) ________________________

shall be secured from the rock source shown on drawing No. ______. All operations within the rock source shall be subject to approval. The contracting authority reserves the right to designate the locations of excavations within the limits of the rock source in order to obtain suitable rock materials for construction purposes. The portions of the rock source to be excavated shall be cleared as provided in section G–2 (Clearing) and shall be stripped of all overburden and loose, soft, disintegrated rock as directed.

The contractor shall produce, by excavation in rock source and selection of processing, sufficient suitable rock fragments reasonably well graded, as determined by the contracting authority, up to ______ inches in maximum dimensions for construction of *(rock fines fill in dam embankment, pervious backfill, and bedding for riprap). The contractor shall also produce by excavation in rock source and selection or processing sufficient suitable rock fragments reasonably well graded, as determined by the contracting authority, up to ______ cubic yards in volume for construction of *(rockfill in dam embankment and riprap). The type of equipment used and the contractor's operations in the rock source shall be such as will produce the required gradations of rock fragments at the rock source.

All suitable rock fragments shall be transported to points of final use, and all excavated materials unsuitable or in excess of requirements for construction purposes shall be disposed of in excavations in rock source or as directed.

[8] Designate rock types, for example granite and limestone.

*Revise or delete as appropriate.

The cost of all work described in this section including clearing and stripping rock source shall be included in the unit prices per cubic yard bid in the schedule for items of construction in which the rock fragments are used.

G-19. ***Disposal of Excavated Material.***—So far as practicable, as determined by the contracting authority, all suitable materials from excavation required under these specifications shall be used in the permanent construction as provided in subsection (e) of section G-6 (Open-Cut Excavation, General). Excavated materials that are unsuitable for or are in excess of permanent construction requirements shall be wasted. The disposal of all excavated materials that are to be wasted shall be subject to the approval of the contracting authority, but the contractor will not be required to haul materials to be wasted more than ______ feet along the most practicable routes to the designated disposal areas.

Waste piles shall be located where they will not interfere harmfully with the natural flow of the stream, with the operation of the reservoir, or with the flow of water to or from the spillway or outlet works, and where they will neither detract from the appearance of the completed project nor interfere with the accessibility of the structures for operation. *(Areas designated for disposal of waste material from excavation are shown on drawings No. ______ and ______.) Where required, waste piles shall be leveled and trimmed to reasonably regular lines.

The cost of transporting excavated materials from excavations to disposal areas or to points of final use, including stockpiling and rehandling, if required, and of disposing of all excavated materials that are wasted, as provided in this section, shall be included in the unit prices per cubic yard bid in the schedule for excavation.

B. EMBANKMENT

G-20. ***Embankment Construction, General.***—For the purpose of these specifications, the term "dam embankment" includes all portions of the dam embankment as follows:[9]

(1) The earthfill portions designated on the drawings.

(2) The sand and gravel fill portions designated on the drawings.

(3) The sand, gravel, and cobble fill portions designated on the drawings.

(4) The miscellaneous fill portions designated on the drawings.

(5) The rockfill portions designated on the drawings.

(6) The crushed-rock or gravel bedding for riprap and the riprap on the upstream slope of the dam embankment.

(7) The seeded topsoil cover on downstream slope of the dam embankment.

(8) Selected surfacing on the crest of the dam embankment.

The embankment shall be constructed to the lines and grades shown on the drawings: *Provided*, That the slopes of the division lines between zones or portions of the embankment are tentative and shall be subject to variation, at any time prior to or during construction, and the contractor shall be entitled to no additional allowance above the unit prices bid in the schedule by reason of such variation. No brush, roots, sod, or other perishable or unsuitable materials shall be placed in the embankment. The suitability of each part of the foundation for placing embankment materials thereon and of all materials for use in embankment construction will be determined by the contracting authority. No embankment material shall be placed in the embankment when either the material or the foundation or embankment on which it would be placed is frozen. The contractor shall maintain the embankment in an approved manner until the final completion and acceptance of all of the work under the contract. The embankment for each portion shall be maintained approximately level throughout the entire length of each layer from abutment to abutment. All openings through the dam embankment required for construction purposes shall be subject to approval, and such openings, if approved, shall be constructed so that the slope of the bonding surface between embankment in place and embank-

[9] List should be revised as applicable.

*Revise or delete as appropriate.

ment to be placed is not steeper than 4:1. The bonding surface of the embankment in place shall be prepared as provided for embankment foundations.

Each load of the material placed in the embankment, whether from excavation for other parts of the work or from borrow pits, shall be placed in the location designated by the contracting authority *(regardless of the classification of the excavation) and the contractor shall be entitled to no additional allowance above the unit prices bid in the schedule on account of this requirement. All portions of the embankment, whether constructed of materials excavated for other parts of the work or from borrow pits, will be measured and paid for in embankment after being compacted, if compacting is required. *(Except for riprap on the upstream slope of the dam embankment and for ____________________), the payment for placing, moistening, and compacting will be in addition to the payment made for the excavation, *(separation), and transportation of the required materials. Payment for excavation and transportation of materials shall include the cost of rehandling excavated materials deposited temporarily in stockpiles. It may be feasible to transport a *(large) portion of the materials which are excavated for other parts of the work and which are suitable for embankment construction, directly to the embankment at the time of making the excavations, but the contractor shall be entitled to no additional compensation above the unit prices bid in the schedule by reason of it being necessary, as determined by the contracting authority, that such excavated materials be deposited in temporary stockpiles prior to being placed in the embankment.

G-21. *Earthfill in Dam Embankment.*—(a) *General.*—The earthfill portions of the dam embankment and earthfill placed for structure foundations ____________________ shall be constructed in accordance with the provisions of this section.

(b) *Preparation of Foundations.*—No material shall be placed in any section of the earthfill portions of the dam embankment until the foundation for that section has been dewatered and suitably prepared and has been approved by the contracting authority. All portions of excavations made for test pits or other subsurface investigations, and all other existing cavities found within the area to be covered by earthfill which extend below the established lines of excavation for dam embankment foundation, shall be filled with compacted earthfill material as herein specified for earthfill in embankment, and payment therefor will be made as provided for earthfill in subsection (h). The foundation, *(except rock, shale, and ____________ surfaces), for the earthfill shall be prepared by leveling and rolling so that the surface materials of the foundation will be as compact and well bonded with the first layer of the earthfill as herein specified for the subsequent layers of the earthfill. *(All shale and ____________ foundation surfaces shall be protected from air-slaking and freezing by leaving ______ feet temporary cover of unexcavated material. Finish excavation shall be made to remove such temporary cover, and rock, shale, and ____________.) Surfaces upon or against which the earthfill portions of the dam embankment are to be placed shall be cleaned of all loose and objectionable materials in an approved manner by hand work or other effective means immediately prior to placing the first layer of earthfill. The surfaces of each portion of the foundation, immediately prior to placing the earthfill, shall have all water removed from depressions and shall be properly moistened and sufficiently clean to obtain a suitable bond with the earthfill.

(c) *Materials.*—The earthfill portion(s) of the dam embankment shall consist of a mixture of the ____________, ____________, and __________ available from excavations required for the dam and appurtenant works or from borrow pits in borrow areas ______, ______, and ______.

The contractor's operations in the excavation of the materials for the earthfill shall be in accordance with section G-16 (Borrow Areas).

(d) *Moisture Control.*—The water content of the earthfill material prior to and during compaction shall be distributed uniformly throughout each layer of the material. The allowable ranges of placement water content are based on design considerations. In general, the average placement water content will be required to be maintained at the Proctor laboratory standard optimum condition. This standard optimum water content is defined as, "That water content which will result in a maximum dry unit weight of the soil when subjected to the Proctor compaction test."

*Revise or delete as appropriate.

The Proctor compaction tests will be made by the contracting authority. The tests can be made by the Bureau of Reclamation procedure using a $\frac{1}{20}$-cubic-foot compaction mold, or by ASTM Designation D 698-42T or the standard AASHO T 99-49 method, both using a $\frac{1}{30}$-cubic-foot compaction mold.

As far as practicable, the material shall be brought to the proper water content in the borrow pit before excavation, as provided in subsection (e) of section G 16 (Borrow Areas). Supplementary water, if required, shall be added to the material by sprinkling on the earthfill and shall be mixed uniformly throughout the layer.

(e) *Placing.*—The distribution and gradation of the materials throughout the earthfill shall be as directed, and shall be such that the fills will be free from lenses, pockets, streaks, or layers of material differing substantially in texture or gradation from the surrounding material. The combined excavation and placing operations shall be such that the materials when compacted in the earthfill will be blended sufficiently to secure the best practicable degree of compaction and stability. Successive loads of material shall be dumped on the earthfill so as to produce the best practicable distribution of the material, subject to the approval of the contracting authority, and for this purpose the contracting authority may designate the locations in the earthfill where the individual loads shall be deposited. The most impervious materials shall be placed in the central upstream portion of the earthfill and the more pervious materials shall be placed so that the permeability of the fill will be gradually increased toward the upstream and downstream slopes of the earthfill.

Cobbles and rock fragments having maximum dimensions of more than 5 inches shall not be placed in the earthfill. Should cobbles and rock fragments of such size be found in otherwise approved earthfill materials, they shall be removed by the contractor either at the site of excavation, *(at the separation plant), or after being transported to the earthfill but before the materials in the earthfill are rolled and compacted. Such cobbles and rock fragments shall be placed in the ________ or ________ portion(s) of the dam embankment or wasted as approved by the contracting authority. The material shall be placed in the earthfill in continuous, approximately horizontal layers not more than 6 inches in thickness after being rolled as herein specified.

If, in the opinion of the contracting authority, the surface of the prepared foundation or the rolled surface of any layer of earthfill is too dry or smooth to bond properly with the layer of material to be placed thereon, it shall be moistened and/or worked with harrow, scarifier, or other suitable equipment, in an approved manner to a sufficient depth to provide a satisfactory bonding surface before the next succeeding layer of earthfill material is placed. If, in the opinion of the contracting authority, the rolled surface of any layer of the earthfill in place is too wet for proper compaction of the layer of earthfill material to be placed thereon, it shall be removed; allowed to dry; or be worked with harrow, scarifier, or other suitable equipment to reduce the water content to the required amount; and then it shall be recompacted before the next succeeding layer of earthfill material is placed. *(The earthfill on each side of the concrete cutoff walls shall be kept at approximately the same level as the placing of the earthfill progresses, and the walls shall be protected carefully against displacement or other damage.)

(f) *Rollers.*—Tamping rollers shall be used for compacting the earthfill. The rollers shall be furnished by the contractor and shall meet the following requirements:

(1) *Roller drums.*—Each drum of a roller shall have an outside diameter of not less than 5 feet and shall be not less than 4 feet nor more than 6 feet in length. The space between two adjacent drums, when on a level surface, shall be not less than 12 inches nor more than 15 inches. Each drum shall be free to pivot about an axis parallel to the direction of travel. Each drum shall be equipped with a suitable pressure-relief valve.

(2) *Tamping feet.*—At least one tamping foot shall be provided for each 100 square inches of drum surface. The space measured on the surface of the drum, between the centers of any two adjacent tamping feet, shall be not less than 9 inches. The length of each tamping foot from the outside surface of the drum shall be maintained at not less than 9 inches. The cross-sectional area

*Revise or delete as appropriate.

of each tamping foot shall be not more than 10 square inches at a plane normal to the axis of the shank 6 inches from the drum surface, and shall be maintained at not less than 7 square inches nor more than 10 square inches at a plane normal to the axis of the shank 8 inches from the drum surface.

(3) *Roller weight.*—The weight of a roller when fully loaded shall be not less than 4,000 pounds per foot of length of drum.

The loading used in the roller drums and operation of the rollers shall be as required to obtain the desired compaction. If more than one roller is used on any one layer of fill, all rollers so used shall be of the same type and essentially of the same dimensions, as determined by the contracting authority. Tractors used for pulling rollers shall have sufficient power to pull the rollers satisfactorily when drums are fully loaded with sand and water. During the operation of rolling, the contractor shall keep the spaces between the tamping feet clear of materials which would impair the effectiveness of the tamping rollers.

(g) *Rolling.*—When each layer of material has been conditioned to have the required moisture, as provided in subsection (d), it shall be compacted by passing the tamping roller, as specified above, over it 12 times. If the water content is less than that required, the rolling shall not proceed except with the specific approval of the contracting authority, and, in that event, additional rolling shall be done, as directed by the contracting authority, to obtain the required compaction, and no adjustment in price will be made therefor. If the water content is greater than that required, the material may be removed and stockpiled for later use or the rolling shall be delayed until such time as the material has dried so that it contains only the required water content, and no adjustment in price will be made on account of any operation of the contractor in removing and stockpiling or in drying the materials. If, with the required water content, it is found desirable to roll each 6-inch layer more or less than 12 times to obtain the required compaction, the number of rollings shall be changed accordingly, as directed by the contracting authority, and adjustment will be made in the unit price bid for earthfill in dam embankment in the amount of ------ cent per cubic yard for each additional or lesser number of rollings required.

(h) *Measurement and Payment.*—Measurement, for payment, of earthfill in dam embankment will be made of all earthfill compacted in place by rollers specified in subsection (f) to the established lines and grades, including refill of additional excavation for structure foundations ordered by the contracting authority. Payment for earthfill in dam embankment will be made at the unit price per cubic yard bid therefor in the schedule, which unit price shall include the cost of placing, wetting, and roller compacting the earthfill material in the dam embankment: *Provided*, That an adjustment of the unit price bid will be made in each case in which it is required that the contractor make more or less than 12 roller passes as provided in subsection (g). Where portions of the earthfill in dam embankment require special compaction, payment therefor will be made as provided in section G–22 (Specially Compacted Earthfill).

G–22. *Specially Compacted Earthfill.*—Where compaction of earthfill material by means of the roller specified for use on the dam embankment is impracticable or undesirable, the earthfill shall be specially compacted as specified herein at the following locations:

(1) Portions of the earthfill in dam embankment adjacent to structures, and structure and embankment foundations shown on the drawings as specially compacted earthfill.

(2) Portions of the earthfill in dam embankment at steep and irregular abutments where designated by the contracting authority.

(3) Earthfill placed to refill additional excavation in common excavation for structure foundations.

(4) Earthfill at locations designated by the contracting authority outside the limits of the dam embankment.

Specially compacted earthfill material shall conform to and be placed in accordance with the applicable provisions of subsections (b), (c), and (e) of section G–21 (Earthfill in Dam Embankment). When each layer of material has been conditioned to have the required water content it shall be compacted by special rollers, mechanical tampers, or other approved methods; and all equipment and methods used shall be subject to

approval. The water control and compaction shall be equivalent to that obtained in the earthfill actually placed in the dam embankment in accordance with subsections (d) and (g) of section G–21.

Measurement, for payment, of specially compacted earthfill will be made of the material specially compacted, as provided in this section, to the established lines and grades and as shown on the drawings. Measurement, for payment, of specially compacted earthfill at steep and irregular dam abutments shown under (b) above, will be limited to a thickness of 2 feet measured horizontally from the average contacts where practicable, or as otherwise determined by the contracting authority. Payment for specially compacted earthfill will be made at the unit price per cubic yard bid therefor in the schedule, which unit price shall include the cost of placing, moistening, and specially compacting the earthfill material. Payment for excavation and transportation of the material used in specially compacted earthfill will be made at the unit price per cubic yard bid in the schedule for excavation of the material.

G–23. *Sand, Gravel, and Cobble Fill in Dam Embankment.*—The sand, gravel, and cobble fill portion(s) of the dam embankment shall be constructed of selected material from excavation for permanent construction required under these specifications and suitable material from borrow area _____. The material shall consist of suitable mixtures of sand, gravel, and cobbles and shall be placed in approximately horizontal layers not to exceed 12 inches in thickness after compaction by four passes of the treads of a crawler type tractor weighing approximately 40,000 pounds. One pass of the treads is defined as the required number of successive tractor trips which, by means of sufficient overlap, will insure complete coverage of an entire 12-inch layer by the tractor treads. Second and subsequent passes of the treads shall not be made until each pass, as defined above, is completed. If boulders are present in the material, they shall be embedded into the dam embankment so as not to interfere with compaction. During or immediately prior to compaction, the material shall be thoroughly wetted. The outer slopes shall be reasonably true to the lines and grades shown on the drawings or established by the contracting authority. The contractor shall be entitled to no additional allowance above the unit prices bid in the schedule on account of delays caused by poor trafficability in borrow area _____ or on the dam embankment or any other difficulties due to overly wet material.

The item of the schedule for sand, gravel and cobble fill in dam embankment constructed of material from required excavation, includes only the placing, wetting and compacting of selected materials from excavation for permanent construction required under these specifications. The item of the schedule for excavating material in borrow area _____ and constructing sand, gravel, and cobble fill in dam embankment, includes excavating suitable material in borrow area _____, transporting such material from the borrow area to the dam embankment, and placing, wetting, and compacting such materials in designated areas of the dam embankment. Materials covered by each of the above items shall be placed in separate designated areas of the dam embankment established by the contracting authority to facilitate measurement.

Measurement, for payment, of sand, gravel, and cobble fill in dam embankment, will be made separately of the material covered by each of the above items compacted in place in the separate designated areas established by the contracting authority. Payment for sand, gravel, and cobble fill in dam embankment constructed of material from required excavation, will be made at the unit price per cubic yard bid therefor in the schedule. Payment for excavation and transportation of the above material will be made at the applicable unit prices per cubic yard bid in the schedule for excavation for the permanent construction involved. Payment for excavating material in borrow area _____ and constructing sand, gravel, and cobble fill in dam embankment will be made at the unit price per cubic yard bid therefor in the schedule.

G–24. *Miscellaneous Fill in Dam Embankment.*—The miscellaneous fill portion of the dam embankment shall be constructed as herein specified of selected material from excavation for permanent construction required under these specifications. The material shall be placed in approximately horizontal layers not exceeding _____ inches in thickness before compaction. Successive loads of the material shall be dumped so as to secure the best practicable distribution

as determined by the contracting authority. The contractor shall route his hauling equipment over each layer of fill material as placed, and distribute the travel evenly over the entire width of the fill so as to obtain the maximum amount of compaction practicable. The material shall be sufficiently moist to prevent dusty conditions and sufficiently dry to prevent rutting. The outer slope of the miscellaneous fill shall conform to the lines and grades shown on the drawings or established by the contracting authority.

Measurement, for payment, of miscellaneous fill in dam embankment will be made of the material compacted in place in the embankment to the lines and grades shown on the drawings or established by the contracting authority. Payment for miscellaneous fill in dam embankment will be made at the unit price per cubic yard bid therefor in the schedule, which unit price shall include the cost of placing, moistening, and compacting the material, and all other operations on the embankment necessary for the completion of the miscellaneous fill as herein specified.

G–25. ***Rockfill in Dam Embankment (Earthfill Dam).***—The rockfill portion(s) of the dam embankment as shown on the drawings shall be constructed as herein specified of rock fragments from excavation for permanent construction and from rock source. The rockfill material shall consist of rock fragments reasonably well graded, as determined by the contracting authority, up to 1 cubic yard in volume. Successive loads of material shall be dumped so as to secure the best practicable distribution of the materials as determined by the contracting authority. To the extent practicable, the larger rock fragments shall be placed on the outer slopes, and the smaller rock fragments shall be placed next to the inner portions of the dam embankment. Rockfill shall be placed in approximately horizontal layers not exceeding 3 feet in thickness. The rock fragments need not be hand placed but shall be dumped and roughly leveled, in a manner to maintain a reasonably uniform surface and insure that the completed fill will be stable and that there will be no large unfilled spaces within the fill.

Measurement, for payment, of rockfill in dam embankment will be made of the rockfill in place in dam embankment to the lines and grades shown on the drawings or established by the contracting authority. Payment for rockfill in dam embankment will be made at the unit price per cubic yard bid therefor in the schedule, which unit price shall include the entire cost of placing the rock fragments in dam embankment. Payment for excavation and transportation of the rock fragments will be made at the unit prices per cubic yard bid in the schedule for excavation of the materials used.

G–26. ***Riprap.***—A layer of riprap ____ feet in thickness shall be placed at locations shown on the drawings and elsewhere as directed. All riprap shall be obtained from the riprap source shown on drawing No. ______. All operations within the riprap source shall be subject to approval. The contracting authority reserves the right to designate the locations of excavations within the limits of the riprap source in order to obtain suitable rock materials for construction purposes. The portions of the riprap source to be excavated shall be cleared as provided in section G–2 (Clearing) and shall be stripped of all overburden and loose, soft, disintegrated rock as directed.

The contractor shall produce by excavation in riprap source and selection or processing sufficient suitable rock fragments reasonably well graded, as determined by the contracting authority, up to ______ pounds in weight for use in riprap. The type of equipment used and the contractor's operations in the riprap source shall be such as will produce the required gradations of rock fragments.

All suitable rock fragments shall be transported to points of final use, and all excavated materials unsuitable or in excess of requirements for construction purposes shall be disposed of in excavations in riprap source or in other areas within the limits of the riprap source as directed.

The rock fragments in riprap need not be compacted but shall be dumped and graded off in a manner to insure that the larger rock fragments are uniformly distributed and that the smaller rock fragments serve to fill the spaces between the larger rock fragments in such a manner as will result in compact uniform layers of riprap of the specified thicknesses. Hand placing will be required only to the extent necessary to secure the results specified above.

Measurement, for payment, of riprap will be made of the riprap in place to the lines shown on the drawings or as established by the contracting authority and on the basis of the prescribed thicknesses. Payment for procuring and placing riprap will be made at the unit price per

cubic yard bid therefor in the schedule, which unit price shall include the cost of producing the riprap, including developing haul road, clearing and stripping riprap source, and excavating, transporting and placing the riprap.

G–27. *Bedding for Riprap.* A layer of bedding for riprap ______ inches in thickness shall be placed at locations shown on the drawings or established by the contracting authority. The bedding material shall consist of rock fragments from rock source, reasonably well graded up to 6 inches in maximum dimension. The material shall be spread without segregation over the designated areas in a uniform layer ______ inches in thickness.

Measurement, for payment, of bedding for riprap will be made of the bedding placed to the established lines and grades on the basis of the prescribed thickness. Payment for procuring and placing rock material for bedding for riprap will be made at the unit price per cubic yard bid in the schedule for bedding for riprap, which unit price shall include the cost of work performed in rock source to produce the rock fragments used in bedding and of transporting and placing the rock fragments in bedding for riprap.

G–28. *Backfill.* Backfill shall be placed ______ ______ and elsewhere as shown on the drawings or as directed. The materials to be used for backfill shall be obtained from excavation for the dam and appurtenant works, or from borrow pits, as directed. The material used for backfill, the amount thereof, and the manner of placing shall be subject to approval.

Measurement, for payment, of backfill will be made of the material in place about the structure to the prescribed lines, grades, and dimensions. Payment for backfill will be made at the unit price per cubic yard bid therefor in the schedule, which unit price shall include the cost of all work connected therewith, except the excavation and compaction of the backfill materials. Where compaction of backfill is required, the compacting shall be performed as provided in section G–29 (Compacting Backfill). Payment for compacting backfill will be made at the unit price per cubic yard bid therefor in the schedule which payment will be in addition to the payment for backfill.

G–29. *Compacting Backfill.* (a) *General.*—Where compacting of backfill is required the materials shall be deposited in horizontal layers and compacted as specified in this section. The distribution of materials shall be such that the compacted material will be homogeneous and free from lenses, pockets, streaks, or other imperfections.

(b) *Compacting Impervious Material.* Where compacting of impervious material is required the material shall be deposited in horizontal layers not more than 6 inches thick after being compacted. The excavating and placing operations shall be such that the material when compacted will be blended sufficiently to secure the best practicable degree of compaction, impermeability, and stability. Prior to and during compaction operations, the material shall have the optimum moisture content required for the purpose of compaction, as determined by the contracting authority, and the moisture content shall be uniform throughout each layer.

Insofar as practicable, as determined by the contracting authority, moistening of the material shall be performed at the site of excavation but such moistening shall be supplemented by sprinkling at the site of compaction if necessary. If the water content is less than optimum for compaction, the compaction operations shall not proceed, except with the specific approval of the contracting authority, and if the water content is greater than optimum for compaction, the compaction operations shall be delayed until such time as the material has dried to the optimum water content, and no adjustment in price will be made on account of any operation of the contractor in drying the material or on account of delays occasioned thereby.

When the material has been conditioned as hereinbefore specified, it shall be compacted by tamping rollers having staggered and uniformly spaced knobs and of sufficient weight for proper compaction, by hand or power tampers, or by other means or equipment approved by the contracting authority. When tamping rollers are used, the tamping knobs and cleaner bars shall be properly maintained and the spaces between the tamping feet shall be kept clear of material which impairs the effectiveness of the tamping roller.

The density (dry) of the soil fraction in the compacted material shall not be less than 95 per-

cent of the laboratory standard maximum soil density (dry) as determined by the Proctor compaction test for the materials being compacted.

(1) *Compaction test.*—The compaction tests will be made by the contracting authority. The standard maximum soil density is the dry weight per cubic foot of the soil compacted at optimum moisture content by laboratory procedure. The compaction test can be made by the Bureau of Reclamation procedure using a $\frac{1}{20}$-cubic-foot compaction mold, or by ASTM Designation D 698–42T or the standard AASHO T 99–49 method, both using a $\frac{1}{30}$-cubic-foot compaction mold.

(c) *Compacting Cohesionless Free-Draining Material.*—Where compacting of cohesionless free-draining materials, such as sands and gravels, is required, the material shall be deposited in horizontal layers and compacted to the relative density specified below.

The thickness of the horizontal layers after compaction shall not be more than 6 inches if compaction is performed by tampers or rollers, not more than 12 inches if compaction is performed by treads of crawler-type tractors, surface vibrators, or similar equipment, and not more than the penetrating depth of the vibrator if compaction is performed by internal vibrators.

The relative density of the compacted material shall be not less than 70 percent as determined by the standard Bureau of Reclamation relative density tests for cohesionless free-draining soils (sec. 115(f)).

(1) *Relative density test.*—The relative density tests will be made by the contracting authority. The relative density of a cohesionless free-draining soil, expressed as a percentage, is defined as its state of compactness with respect to the loosest and most compact states at which it can be placed by laboratory procedures. The relative density will be based on the following formula, wherein the maximum density is the highest unit weight of the soil, minimum density is the lowest unit weight of the soil, and in-place density is the unit weight of the soil in-place. Tests for moisture content are made on the materials and the unit weights are expressed in terms of ovendry weights.

$$\left[\begin{matrix}\text{Relative}\\ \text{density (\%)}\end{matrix}\right]=\frac{\left[\begin{matrix}\text{maximum}\\ \text{density}\end{matrix}\right]\times\left[\left(\begin{matrix}\text{in-place}\\ \text{density}\end{matrix}\right)-\left(\begin{matrix}\text{minimum}\\ \text{density}\end{matrix}\right)\right]}{\left[\begin{matrix}\text{in-place}\\ \text{density}\end{matrix}\right]\times\left[\left(\begin{matrix}\text{maximum}\\ \text{density}\end{matrix}\right)-\left(\begin{matrix}\text{minimum}\\ \text{density}\end{matrix}\right)\right]}\times 100$$

(d) *Cost.*—The cost of compacting backfill as described in this section shall be included in the unit price bid in the schedule for compacting backfill.

G–30. ***Filters.***—Graded sand and gravel filters shall be constructed under the *(apron, weir, spillway floor lining) as shown on the drawings or as directed. All materials for the filters shall be furnished by the contractor.

Trenches for the filters shall be excavated to lines, shapes, and dimensions shown on the drawings. Overexcavation in a manner to disturb the compacted foundations will not be permitted, and any material outside of the required lines which is disturbed shall be removed, and shall be replaced at the expense of the contractor in the manner described in section G–6 (Open-Cut Excavation, General). The sand and gravel shall be placed and tamped into place in such a manner that mixing of sand with gravel in the filter or with foundation or backfill materials will not occur. The graded sand and gravel shall be placed and tamped to the dimensions shown. *(After the graded sand and gravel in the filter have been shaped and compacted to the required depths, surfaces of the filter over which concrete is to be placed shall be covered with a layer of mortar 1 inch thick to provide a covering that will prevent the filter material from being displaced during the placing of the concrete. The mortar coating shall be applied carefully to the required thickness. The consistency of the mortar and methods of application shall be such as to avoid unnecessary filling of the voids in the filter material.)

Materials for filters shall be as follows:

(1) Gravel under the ________________ shall be clean, well-graded gravel from $\frac{3}{16}$ inch to $1\frac{1}{2}$ inches in size.

(2) Sand shall conform to the requirements specified for concrete in section G–49 (Sand).

*Revise or delete as appropriate.

Measurement, for payment, of graded sand and gravel in filters will be of the volume of sand and gravel in the completed filter. Payment for graded sand and gravel for filters will be made at the unit price per cubic yard bid therefor in the schedule, which price shall include the cost of procuring, delivery, handling, placing, and compacting the graded sand and gravel *(and furnishing sand for and mixing and placing the mortar covering). Excavation for filters will be paid for in accordance with subsection (b) of section G-6 (Open-Cut Excavation, General).

C. PRESSURE GROUTING

G-31. ***Requirement for Grouting.***—The contractor shall drill and grout under pressure:

(1) The rock foundation of the dam embankment;

(2) The rock surrounding the portion(s) of the tunnel; and

(3) The rock formations and structures at other locations as shown on the drawings, or as directed.

The contractor shall grout under pressure:

*(1) The joint between the first-stage and second-stage concrete of the diversion inlet plug in the outlet-works intake structure; and

*(2) The joints between the first-stage and second-stage concrete in the outlet-works gate structure.

The amount of drilling and pressure grouting that will be required is uncertain and the contractor shall be entitled to no additional compensation above the unit prices bid in the schedule by reason of increase or decrease of the schedule quantities except as provided herein. The grouting shall be done either prior to or after the placing of any part or all of the concrete as directed or as prescribed herein. *(The joints between first- and second-stage concrete in the outlet-works intake structure, and in the outlet-works gate structure shall be grouted after the second-stage concrete has set and cooled to the satisfaction of the contracting authority.)

The items in the schedule for which payment will be made for any grouting operation do not include the work necessary for drilling holes, furnishing and placing pipe, hookup to grout holes and connections, and any other work necessary in the placing of mortar or grout by the grouting method.

G-32. ***Drilling Grout Holes.***—Grout holes shall be drilled into the rock at the locations as described in section G-31 (Requirement for Grouting). As the construction work progresses, the development of leakage, the condition of the surrounding rock, or the results of grouting operations may indicate that the rock already covered or lined with concrete requires grouting, in which event holes shall be drilled through the concrete, including concrete in grout cap, and into the underlying or surrounding rock, and pipes for grout connections shall be placed as directed. The location, direction, order of drilling and depth of each hole shall be as shown on the drawings or as directed. The holes shall be drilled with rotary-type drills and the use of rod dope, grease, or other lubricants on the drill rods or in the grout holes will not be permitted. Drilling grout holes with percussion-type drill will not be permitted, except for installing pipe nipples forming the collars of the grout holes when needed and for placing mortar or grout by the grouting method. The requirements as to depth, spacing, and direction of holes are approximate and subject to revision during the work of drilling, testing, and grouting. It is expected that the required depth of holes will not exceed ____ feet.

The minimum diameter of each grout hole shall be not less than that produced by the Commercial Standard EX size bit, approximately $1\frac{1}{2}$ inches. Unless otherwise directed, the first holes in the grout cap or in the foundation rock, and the first rings of grout holes and the grout holes within each grout ring in the *tunnels and *shafts, shall be spaced widely and shall be drilled and grouted before the intermediate holes are drilled and grouted. These intermediate holes shall be drilled

*Revise or delete as appropriate

and grouted before further intermediate holes are drilled and grouted, and in this manner the drilling and grouting of all holes and rings and the holes within each ring shall be completed with such final spacing of holes and rings as the grouting results show to be necessary. After holes in a region have been drilled and grouted, it may be found necessary to drill additional grout holes. No allowance above the unit prices bid in the schedule will be made for the drilling of such holes or for the expense of moving equipment to other operations and returning to a previously drilled area.

Wherever required, the drilling and grouting shall be performed in successive operations consisting in each case of drilling the hole to a limited depth, grouting at that depth, cleaning out the grout hole by washing or other suitable means before the grout in the hole has set sufficiently to require redrilling, allowing the grout surrounding the grout hole to attain its initial set, drilling the hole to an additional depth and then grouting, and thus successively drilling and grouting the hole at various depths within the stages until the required depth of hole is completely drilled and grouted, all as determined by the contracting authority. Redrilling required because of the contractor's failure to clean out a hole before the grout has set shall be performed at the contractor's expense. When grout has been allowed to set in a hole by direction of the contracting authority, the required redrilling will be paid for at the rate of 50 percent of the unit price per linear foot bid in the schedule for drilling grout holes in stage between depth of 0 foot and 30 feet regardless of depth. No additional allowance above the unit prices bid in the schedule for drilling grout holes in stage will be made on account of the requirement for cleaning out holes before further drilling, or on account of moving of equipment that may be necessary due to the requirement for such successive shallow depth and deeper drilling and grouting.

When the drilling of each hole has been completed, it shall be protected from becoming clogged or obstructed by being capped temporarily or otherwise protected suitably until it is grouted, and any hole that becomes obstructed before it is grouted shall be opened completely at the expense of and by the contractor. Prior to drilling for grouting the rock surrounding the *tunnels and *shafts, the contractor shall complete the work of placing mortar or grout by the grouting method. Measurement, for payment, of drilling grout holes will be made only of the linear feet of holes actually drilled in concrete or rock as directed. Measurement, for payment, for determining stage depths of drilling grout holes in rock and concrete will be made from the collar of the hole at the exposed surface of the rock or concrete to the actual depth drilled into the rock foundation and concrete as directed.

Except as provided in section G–31 (Requirement for Grouting), payment for drilling grout holes in rock or in concrete will be made at the applicable unit price per linear foot bid in the schedule for drilling grout holes in stage between the depths specified in the schedule, which unit prices shall include the cost of furnishing all labor, materials, tools and equipment required for drilling the holes; maintaining the holes free from obstruction until grouted; and all incidental work connected therewith.

Payment for drilling holes for setting foundation grout pipe or expansion-type plugs for grouting the rock surrounding the *tunnels and *shafts will be made at the unit price per linear foot bid in the schedule for drilling grout holes in stage between depths of 0 foot and 30 feet. The cost of drilling holes for placing mortar or grout by the grouting method shall be included in the unit price per cubic yard bid in the schedule for concrete involved.

G–33. ***Pipe for Foundation and Joint Grouting.***—Standard black pipe for foundation and construction joint grouting and for grouting rock surrounding *tunnels and *shafts shall be set in the concrete or rock in the locations described in section G–31 (Requirement for Grouting). Pipes for grouting shall be set over springs, crevices in the rock, faults, or other foundation defects as directed. Grout pipes set in concrete except those in grout caps which may be left projecting above the footings, shall end not less than 3 inches inside the finished surface of the concrete. A standard coupling and wrapped nipple to facilitate removal after grouting shall be attached to the grout pipe and shall extend beyond the finished surface of the concrete as shown on the drawings. The holes left after removal of wrapped nipples

*Revise or delete as appropriate.

and other grout holes shall be filled immediately and completely with drypack in accordance with section G-60 (Repair of Concrete).

The size of the grout pipe for each hole and the depth of the holes for setting pipe for grouting shall be as shown on the drawings or as directed. The grout pipes shall be anchored into the rock or concrete into which they are inserted, and the spaces around the pipes shall be carefully sealed with oakum, grout, or other suitable material to prevent entry of concrete or other foreign materials prior to grouting. All pipe and fittings to be embedded in rock or concrete shall be cleaned thoroughly of all dirt, grease, grout and mortar immediately before being embedded in the rock or concrete. The pipe and fittings shall be carefully assembled and placed and shall be held firmly in position and protected from damage while the concrete is being deposited. Care shall be taken to avoid clogging or obstructing the pipes before being grouted, and any pipe that becomes clogged or obstructed from any cause shall be cleaned satisfactorily or replaced at the expense of and by the contractor.

*(Systems of standard pipe, fittings, and special grouting outlets shall be placed in the joints between the first- and second-stage concrete of the outlet-works gate structure and the diversion inlet in the outlet-works intake structure, as shown on the drawings or as directed by the contracting authority. The special grouting outlets consist of modified electrical conduit boxes as shown on the drawings. The pipe and fittings shall be carefully assembled and placed and shall be held firmly in position and protected from damage while the concrete is being deposited. Care shall be exercised to insure that the two companion members of each conduit-box-cover grouting outlet are maintained in accurate alinement and position with respect to each other and that each member becomes an integral part of and moves with the concrete mass to which it is anchored. The method of attaching the first member of each grouting outlet to the forms and, in turn, the second member to the first shall be as shown on the drawings or as directed. This method shall be adhered to accurately unless it is modified by special instructions of the contracting authority. Care shall also be taken to insure that all parts of the system are maintained free from dirt and any other foreign substance.)

*(After the grouting system is placed and before any concrete is placed around it, and at such other times as the contracting authority may direct, the pipe shall be tested by forcing a current of air or water under pressure through it to the satisfaction of the contracting authority, after which it shall be immediately capped temporarily or otherwise closed to avoid the possibility of any foreign substance entering it until it is pressure grouted. Any pipe that becomes clogged before final acceptance of the work due to any cause, shall, if practicable, be cleaned or opened to the satisfaction of the contracting authority. For any plugged pipe which the contractor fails to open or replace, the contractor shall pay to the contracting authority as fixed, agreed and liquidated damages, the sum of ____ dollars ($____) per linear foot of the total length of pipe which is thereby made ineffective, as determined by the contracting authority.)

All standard black pipe and fittings, special grouting outlets, oakum, asphalt emulsion for sealing purposes, nails, tie wire, and temporary supports required for installation of the foundation grout pipe and the joint grouting systems, shall be furnished by the contractor. The pipe shall be type I, class A, standard black pipe in accordance with Federal Specification WW-P-406a. The pipe fittings shall be malleable iron, type I, in accordance with Federal Specification WW-P-521c. The pipe shall be cut, threaded if necessary, fabricated as required, and placed by the contractor.

Payment for furnishing and placing standard black pipe fittings and special grout outlets for grouting will be made at the unit price per pound bid in the schedule for furnishing and placing pipe and fittings for grouting, which unit price shall include the cost of furnishing, unloading, hauling, storing, handling, and installing pipe, fittings and special grout outlets; of protecting the pipe from injury and clogging; and of removing nipples in exposed faces and filling with drypack-filling the holes left by removal of nipples from other grouting operations: *Provided*, That the cost of furnishing and placing pipe and fittings to place mortar or grout by the grouting method shall be

*Revise or delete as appropriate.

included in the unit prices per cubic yard bid in the schedule for concrete involved. Payment will be made only for pipe and fittings actually installed and left in place as directed by the contracting authority, and no additional allowance above the unit price bid in the schedule will be made on account of the varying size lengths or number of pipes required.

G-34. *Hookup to Grout Holes and Connections.*—Payment will be made under the item of the schedule for hookup to grout holes and connections, only once for each hole or connection actually hooked onto at the direction of the contracting authority regardless of the additional number of times packer settings are made or the same hole is hooked onto, and regardless of the volume of water or grout actually injected in the grout hole or connection.

The number of separate grout holes or connections requiring hookups as shown in the schedule is only approximate, and the contractor shall be entitled to no additional compensation above the unit price bid in the schedule by reason of the number of hookups actually required to complete the grouting operations specified in section G-35 (Pressure Grouting).

Payment for connections to cracks, crevices, or seams in the foundation when determined necessary by the contracting authority *(and connections to grout headers for grouting the joint between the first- and second-stage concrete of the outlet-works gate structure and the diversion inlet plug) will be made in accordance with the preceding provisions under the unit price bid in the schedule for hookup to grout holes and connections.

The cost of hooking to connections, for placing mortar or grout by the grouting method, shall be included in the unit prices per cubic yard bid in the schedule for concrete involved.

G-35. *Pressure Grouting.*—Each drilled grout hole and grout connection for pressure grouting foundations *(and joints), as described in section G-31 (Requirement for Grouting), shall have grout composed of cement and water or cement, sand and water forced into it under pressure. Pressures as high as practicable but which, as determined by trial, are safe against rock or concrete displacement, shall be used in the grouting. The placing of mortar or grout by the grouting method shall be done at pressures not exceeding 30 pounds per square inch as determined by the contracting authority. The proportions of cement, sand and water used in mixing the grout, the time of grouting, the pressures used for grouting, and all other details of the grouting operations shall be as determined by the contracting authority.

Different grouting pressures may be required for grouting different sections of the grout holes. Where such grouting of a hole is directed, the grouting shall be performed by attaching a packer to the end of a grout-supply pipe, lowering the grout-supply pipe into the hole to the top of the bottom section that is required to be grouted, grouting at the required pressure, allowing the packer to remain in place until there is no back pressure, withdrawing the grout-supply pipe and packer to the top of the next higher section that is required to be grouted at a different pressure, and thus successively grouting the hole in sections at the specified grouting pressures until the entire hole is completely grouted, except that the grouting of the top sections shall be performed without the use of a packer. The grout-supply pipes and packers shall be furnished by the contractor. The packers shall consist of expansible tubes or rings of rubber, leather, or other suitable material attached to the end of the grout-supply pipe. The packers shall be designed so that they can be expanded to seal the drill hole at the specified elevations and, when expanded, shall be capable of withstanding without leakage for a period of 5 minutes, water pressure equal to the maximum grout pressures to be used. The amount of packer grouting that will be required will depend upon the conditions disclosed by the drilling of grout holes, and the contractor shall be entitled to no additional allowance above the unit price bid in the schedule for pressure grouting by reason of the amount of such work being required.

All intersected rock crevices, seams, or faults containing clay or other washable materials shall be washed with water and air under pressure to remove as much of such materials as possible. If practicable, as determined by the contracting authority, such materials shall be ejected from one or more holes by introducing water under pressure in an adjacent hole. All grout holes shall be tested with clean water under continuous pressure up to the required grouting pressure in order to

*Revise or delete as appropriate.

clean effectively intersected cracks and seams and to determine volume and extent of leakage.

The apparatus for mixing and placing grout shall be of a type approved by the contracting authority and shall be capable of mixing effectively and stirring the grout and forcing it into the holes or grout connections in a continuous, uninterrupted flow at any specified pressure up to a maximum of 250 pounds per square inch. The mixer shall be mechanically operated and provided with an accurate meter, reading in cubic feet to tenths of a cubic foot, for controlling the amount of mixing water used in the grout. In addition to the grout mixer, holdover mechanical agitator tanks shall be provided. All grout shall be pumped with a duplex piston-type pump or other type of pumping equipment approved by the contracting authority. The grouting equipment shall be maintained in a manner satisfactory to the contracting authority and so as to insure continuous and efficient performance during any grouting operation. The arrangement of the grouting equipment shall be such as to provide a supply line and return line from the grout pump to the grout hole. Provision shall be made to permit continuous circulation and accurate control of grouting pressures and grout flows into the grout holes.

*(Expansion-type plugs, if used for grout hole connections in grouting of tunnels, shafts and adits shall be furnished by the contractor. The plugs shall consist of expansible tubes or rings of rubber, leather, or other suitable material, pipe and fittings. The plugs shall be designed so that they can be expanded to seal the drill holes and, when expanded, shall be capable of withstanding without leakage, water pressure equal to the maximum grout pressures to be used. The cost of furnishing all necessary materials and labor for constructing and handling expansion-type plugs shall be included in the unit price per sack bid in the schedule for pressure grouting.)

If, during the grouting of any hole, grout is found to flow from adjacent grout holes or foundation grout connections in sufficient quantity to interfere seriously with the grouting operation or to cause appreciable loss of grout, such connections may be capped temporarily. When grouting is being done with packers and grout returns from adjacent holes, the pressure of the returning grout shall be measured by seating a packer in the adjacent hole and such pressures shall be kept below the allowable pressures for that stage of that hole. Where such capping is not essential, ungrouted holes shall be left open to facilitate the escape of air and water as the grout is forced into other holes. Before the grout has set, the grout pump shall be connected to adjacent capped holes and to other holes from which grout flow was observed, and grouting of all holes shall be completed at the pressures specified for grouting. If, during the grouting of any hole, grout is found to flow from points in the foundations, abutments, or any parts of the concrete structures, or other locations, such flows or leaks shall be plugged or calked by the contractor as directed.

The grouting of any hole shall be continued until the hole or grout connection takes grout at the rate of less than 1 cubic foot of the grout mixture in 20 minutes if pressures of 50 pounds per square inch or less are being used, in 15 minutes if pressures between 50 and 100 pounds per square inch are being used, in 10 minutes if pressures between 100 and 200 pounds per square inch are being used, and in 5 minutes if pressures in excess of 200 pounds per square inch are being used. So far as practicable, the full grouting pressures shall be maintained constantly during grout injections. However, as a safeguard against rock or concrete displacement or while grout leaks are being calked, the contracting authority may require the reduction of the pumping pressure or the discontinuance of pumping. After the grouting of the holes or connections is completed, the pressure shall be maintained by means of stopcocks, or other suitable valve devices, until the grout has set sufficiently so that it will be retained in the holes or connections being grouted.

*(As soon as possible after the second-stage concrete in the outlet-works gate structure and the diversion inlet plug of the outlet-works intake structure has cooled the desired amount, as determined by the contracting authority, the joints between the first-stage concrete and the second-stage concrete shall be pressure grouted with cement grout. The grouting of each joint shall be completed before the grout takes its set in the grout pipe systems, but shall not be grouted so rapidly that the grout will not settle in the joint, and in no case shall the time consumed in filling each joint be less than 30 minutes. Before each

*Revise or delete as appropriate

484556 O—60——38

joint is grouted, it shall be washed and tested thoroughly with air and water under pressure. The grout shall be pumped into the bottom header of the piping system for each joint and the vent or highest header shall be left open until each joint is filled with grout of proper consistency for retention in the joint, whereupon the header shall be closed and the required pressure applied. All other details shall be in accordance with these specifications and as directed by the contracting authority.)

Measurement, for payment, of pressure grouting will be made on the basis of the number of sacks of cement actually forced into the holes or grout connections at the direction of the contracting authority, or required to fill permanent pipes. In measuring grout for payment, the volume of one sack of cement will be considered as 1 cubic foot. Payment for pressure grouting will be made at the unit price per sack bid therefor in the schedule, which unit price shall include the cost of furnishing all labor, materials, tools, and equipment required for the grouting; including plugging or calking of leaks; use of packers; washing crevices, seams or faults with water and air; and testing grout holes with water, except that payment for furnishing and handling cement will be made at the unit price per barrel bid therefor in the schedule, and payment for hooking onto each foundation grout hole or grout connection will be made as described in section G–34 (Hookup to Grout Holes). No payment will be made for grout or for cement used in grout, lost due to improper anchorage of grout pipes or connections, or rejected by the contracting authority on account of improper mixing, or lost by leakage due to the failure of the contractor to calk surface leaks when directed by the contracting authority. All pressure grouting operations shall be performed in the presence of a duly authorized representative of the contracting authority.

D. CONCRETE SPECIFICATIONS

G–36. *Introduction.*—(a) *General.*—Two different concrete specifications are included herein, one for small quantities of concrete where the structures are relatively simple, and one for more complex work requiring a larger amount of concrete and in which the structures are such that more detailed coverage of the work and the manner in which the work is to be done is desirable. The specifications for work requiring detailed control do not include all the specifications requirements that would be needed for a large mass-concrete dam having, say, 100,000 or more cubic yards of concrete.

Unless stated otherwise, all references to "designations" in part D of this appendix refer to designations listed in the appendix of the Bureau of Reclamation Concrete Manual, sixth edition [1].[10]

(b) *Ready-Mixed Concrete.*—The specifications included herein primarily apply to concrete manufactured at the job site. However, all the requirements also apply to ready-mixed concretes.

[10] Numbers in brackets represent items in the bibliography, sec. G–83.

1. *Concrete Specifications for Small Jobs*

G–37. *Source.*—The following sections have been prepared from guide specifications normally used by the Bureau of Reclamation for less than 200 cubic yards of concrete.

G–38. *Materials.*—Concrete shall be composed of cement, sand and coarse aggregate, water, air-entraining agent, and calcium chloride as required, all well mixed and brought to the proper consistency. In general, cement, air-entraining agent, and sealing compound will be accepted on manufacturer's certification of compliance with specifications requirements. Permission to ship on manufacturer's certification shall in no way relieve the contractor of the responsibility for furnishing materials meeting specifications requirements.

(a) *Cement.*—Cement shall be type ______, low alkali,[11] in accordance with Federal Specifica-

[11] Determine type of cement to be used. If the aggregates to be used in concrete are reactive with alkalies in cement, use of low-alkali cement should be specified. If aggregates are not reactive, do not include this requirement. In areas where the hardened concrete will be in contact with soil or water containing soluble sulfates, a sulfate resisting cement should be specified. Refer to table F-1 (app. F).

tion SS–C–192b for portland cement or with the "Standard Specifications for Portland Cement" of the American Society for Testing Materials, Designation C 150. (NOTE.—One specification or the other should be indicated, not both. Federal specifications are usually required for Government work.) The cement shall be free from lumps and damaged cement when used in concrete.

(b) *Water.*—Water shall be free from objectionable quantities of silt, organic matter, alkalies, salts, or other impurities.

(c) *Sand and Coarse Aggregate.*—Sand and coarse aggregate shall be furnished by the contractor from any approved source. The sand particles shall be hard, dense, durable, uncoated rock fragments that will pass a screen having $^3/_{16}$- or $^1/_4$-inch square openings. The sand shall be well graded from fine to coarse and shall be free from injurious amounts of dirt, organic matter, and other deleterious substances.

The coarse aggregate shall consist of natural gravel or crushed rock or a mixture of natural gravel and crushed rock and shall be hard, dense, durable, uncoated rock fragments, free from injurious amounts of thin pieces, organic matter, or other deleterious substances. The coarse aggregate shall be reasonably well graded from $^3/_{16}$ inch to $1^1/_2$ inches, and shall be separated into two sizes by an intermediate screen having $^3/_4$-inch square openings. Screens having openings of other sizes and shapes may be used, if equivalent results are obtained. (NOTE.—In lieu of the above requirements for sand and coarse aggregate, the specifications could require the aggregates to conform to the "Standard Specifications for Concrete Aggregates" of the American Society for Testing Materials, Designation C 33.)

(d) *Air-Entraining Agent.*—The air-entraining agent shall conform to the provisions of ASTM Designation C 260–58T.

(e) *Reinforcement Bars* *(*and Fabric*).—Reinforcement bars shall conform to Federal Specification QQ–S–632, type II, grade C through grade G: *Provided*, That all bars in structures required to be bent to a radius of bend of 50 diameters or less shall be intermediate-grade billet steel (grade C). *(Fabric shall be electrically welded wire fabric conforming to ASTM Designation A 185.)

*Revise or delete as appropriate.

(f) *Sealing Compound.*—Sealing compound shall be white-pigmented compound conforming to "Tentative Specifications for Liquid Membrane-Forming Compounds for Curing Concrete," ASTM Designation C 309–58.

G–39. *Composition.* Sand and coarse aggregate shall be mixed in proportions as directed by the contracting authority. The concrete shall contain not less than 6 sacks of cement per cubic yard for concrete containing $1^1/_2$-inch-maximum-size aggregate and 7 sacks per cubic yard for concrete containing $^3/_4$-inch-maximum-size aggregate. The contractor shall use 1 percent of calcium chloride, by weight of the cement, in all concrete placed when the weather is cold enough to require protection of the concrete from freezing. (NOTE.—Calcium chloride should not be used in concrete containing type V cement. When type V cement is specified, the requirements for use of calcium chloride are to be deleted.) The slump of the concrete shall not exceed 2 inches for slabs, and 3 inches for all other concrete. Air-entraining agent shall be used in such amount as will effect the entrainment of from 4 to 6 percent of air, by volume, of the concrete as discharged from the mixer. When calcium chloride is being used, the portion of mixing water containing the air-entraining agent shall be introduced separately into the mixer.

G–40. *Batching and Mixing.*—The sand and coarse aggregate shall be weighed and shall be proportioned on the basis of integral sacks of cement unless the cement is weighed. Weighing equipment of the beam type may be used. The mixing time shall be at least $1^1/_2$ minutes. Truck mixers will be permitted only when the mixers and their operation are such that the concrete throughout the mixed batch and from batch to batch is uniform with respect to consistency and grading. Any concrete retained in truck mixers so long as to require additional water to permit satisfactory placing shall be wasted.

G–41. *Forms, Preparations for Placing, and Placing.*[12]—Forms shall be used to shape the concrete to the required lines. The surfaces of construction joints shall be clean and damp when covered with fresh concrete or mortar. Cleaning shall consist of the removal of all laitance, loose or

[12] Additional requirements for forming, preparation for placing, and placing may be appropriate for some small jobs, depending on the type of structure involved. See requirements for these phases of construction work as contained in secs. G–44 through G–62.

defective concrete, coatings, sand, sealing compound if used, and other foreign material.

The methods and equipment used for transporting concrete, and the time that elapses during transportation, shall be such as will not cause appreciable segregation of coarse aggregate or slump loss in excess of 1 inch in the concrete as it is delivered to the work. Retempering of concrete will not be permitted. Concrete shall be vibrated until it has been consolidated to the maximum practicable density, is free from rock pockets of coarse aggregate, and closes snugly against all surfaces of forms and embedded materials. Exposed unformed surfaces of concrete shall be brought to uniform surfaces and worked with suitable tools to a reasonably smooth woodfloat or steel-trowel finish as directed.

G-42. *Protection and Curing.*—The contractor shall protect all concrete against injury until final acceptance by the contracting authority. (NOTE.—The requirements for protection of concrete against freezing, including protection at 50° F. for the first 72 hours after the concrete is placed, as contained in the specifications for large jobs in section G-61 (Protection), will also apply for small jobs.)

The concrete shall be cured by water curing or by membrane curing. If concrete is cured by water curing, the concrete shall be kept continuously moist for at least 14 days after being placed by sprinkling or spraying or by other methods approved by the contracting authority. Membrane curing of concrete shall be by application of sealing compound, and application of the sealing compound shall be in accordance with the procedures contained in the Bureau of Reclamation Concrete Manual [1]. Any concrete found to be damaged or defective, by reason of the contractor's operations, at any time before completion and acceptance of the work, shall be removed and replaced by the contractor with acceptable concrete at no additional cost.

G-43. *Reinforcement.*—Steel reinforcement bars *(and fabric) shall be placed in the concrete where shown on the drawings. Before reinforcement is placed, the surfaces shall be cleaned of heavy flaky rust, loose mill scale, dirt, grease, or other foreign substances. Reinforcement shall be accurately placed and secured in position so that it will not be displaced during the placing of the concrete.

Reinforcement will be inspected for compliance with requirements as to size, shape, length, splicing, position, and amount after it has been placed.

*Revise or delete as appropriate.

2. *Concrete Specifications for Large Jobs*

G-44. *Source.*—The following sections have been prepared from guide specifications normally used by the Bureau of Reclamation for work involving concrete quantities of 200 cubic yards or more, and when quality control is necessary.

G-45. *Composition.*—(a) *General.*—Concrete shall be composed of cement, sand, coarse aggregate, water, and admixtures as specified, all well-mixed and brought to the proper consistency.

(b) *Maximum Size of Aggregate.*—The maximum size of coarse aggregate in concrete for any part of the work shall be the largest of the specified sizes, the use of which is practicable from the standpoint of satisfactory consolidation of the concrete by vibration. Three-inch-maximum-size aggregate shall be used in walls that are 15 inches or more in thickness and in slabs that are 8 inches or more in thickness, except that where reinforcement is unusually congested, an appropriately smaller maximum size of aggregate shall be used. Three-inch-maximum-size aggregate shall also be used in any location where concrete is placed against excavated surfaces and the thickness of the concrete will be sufficiently greater than that shown on the drawings to permit the use of the larger size aggregate. (NOTE.—Delete requirements for 3-inch aggregate when use of this size aggregate is not specified. Normally, use of 3-inch aggregate is appropriate only where the quantity of concrete is sufficiently large to warrant its use from an economic standpoint.)

(c) *Mix Proportions.*—The proportions in which the various ingredients are to be used for different parts of the work shall be as determined from time to time during the progress of the work and as tests are made of samples of the aggregates and the resulting concrete. The mix proportions and appropriate water-cement ratio will be determined on the basis of procuring concrete having suitable workability, density, impermeability, durability, and required strength, without the use of an excessive amount of cement. The net water-cement ratio of the concrete (exclusive of water

absorbed by the aggregates) shall not exceed 0.60 by weight, and water-cement ratios used shall be in accordance with those contained in table F-2 (app. F), except that the water-cement ratios shall be reduced, if necessary, to attain required strengths. Tests of the concrete will be made by the contracting authority, and the mix proportions shall be adjusted whenever necessary for the purpose of securing the required economy, workability, density, impermeability, durability, or strength. The contractor shall be entitled to no additional compensation because of such adjustments.

(d) *Consistency.*—The amount of water used in the concrete shall be regulated as required to secure concrete of the proper consistency and to adjust for any variation in the moisture content or grading of the aggregates as they enter the mixer. Addition of water to compensate for stiffening of the concrete before placing will not be permitted. Uniformity in concrete consistency from batch to batch will be required. The slump of the concrete, after the concrete has been deposited but before it has been consolidated, shall not exceed 2 inches for concrete in the tops of walls, piers, parapets, and curbs, and in slabs that are horizontal or nearly horizontal, 4 inches for concrete in sidewalls and arch of tunnel lining, and 3 inches for all other concrete. The contracting authority reserves the right to require a lesser slump whenever concrete of such lesser slump can be consolidated readily into place by means of the vibration specified in section G-58 (Placing). The use of buckets, chutes, hoppers, or other equipment which will not readily handle and place concrete of such lesser slump, will not be permitted.

(e) *Tests.*—The compressive strength of the concrete will be determined by the contracting authority through the medium of tests of 6- by 12-inch cylinders made and tested in accordance with the Bureau of Reclamation Concrete Manual [1], designations 29 to 33, inclusive, except that, for all concrete samples from which cylinders are to be cast, the pieces of coarse aggregate larger than 1½ inches will be removed by screening or hand picking. Slump tests will be made by the contracting authority in accordance with designation 22. The contractor shall provide such facilities as may be necessary for procuring and handling representative test samples.

G-46. *Cement.*—(a) *General.*—To prevent undue aging of sacked cement after delivery, the contractor shall use sacked cement in the chronological order in which it was delivered on the job. Each shipment of sacked cement shall be stored so that it may readily be distinguished from other shipments. Bins in which bulk cement is stored shall be weathertight. The bins shall be emptied and cleaned by the contractor when so directed; however, the intervals between required cleanings will normally not be less than 4 months. The cement shall be free from lumps and shall be otherwise undamaged when used in concrete. If the cement is delivered in paper sacks, empty paper sacks shall be burned. The cement shall be type _____, low alkali,[13] in accordance with Federal Specification SS-C-192b for portland cement or with the "Standard Specifications for Portland Cement" of the American Society for Testing Materials, Designation C 150. (NOTE.—One specification or the other should be indicated, not both. Federal specifications are usually required for Government work.)

(*b*) *Inspection.*—If directed by the contracting authority, the cement will be sampled and tested by the Government in accordance with Federal Test Method Standard No. 158, or the applicable method of test cited in the "Standard Specifications for Portland Cement" of the American Society for Testing Materials, Designation C 150. (See note above.)

G-47. *Admixtures.*—(a) *Accelerator.*—The contractor shall use 1 percent of calcium chloride, by weight of the cement, in all concrete placed when the mean daily temperature in the vicinity of the work site is lower than 40° F. Calcium chloride shall not be used otherwise, except upon written approval. Requests for such approval shall state the reason for using calcium chloride and the percentage of calcium chloride to be used, and the location of the concrete in which the contractor desires to use the calcium chloride. Calcium chloride shall not be used in excess of 2 percent, by weight, of the cement. Calcium chloride shall be measured accurately and shall be added to the batch in solution in a portion of

[13] Determine type of cement to be used. If the aggregates to be used in concrete are reactive with alkalies in cement, use of low alkali cement should be specified. If aggregates are not reactive, do not include this requirement. In areas where the hardened concrete will be in contact with soil or water containing soluble sulfates, a sulfate resisting cement should be specified. Refer to table F-1 (app. F).

the mixing water. Use of calcium chloride in the concrete shall in no way relieve the contractor of responsibility for compliance with the requirements of these specifications governing protection and curing of the concrete. (NOTE.—Calcium chloride should not be used in concrete containing type V cement. When type V cement is specified, the requirements for use of calcium chloride are to be deleted.)

(b) *Air-Entraining Agents.*—The contractor shall use an air-entraining agent in all concrete. The agent used shall conform to the requirements of ASTM Designation C 260–58T, and shall be of uniform consistency and quality within each container and from shipment to shipment. Agents will be accepted on manufacturer's certification of conformance with specifications, but permission to ship on certification shall in no way relieve the contractor of responsibility for furnishing an agent meeting specifications requirements. Agents shall be subject to sampling and testing. The amount of air-entraining agent used in each concrete mix shall be such as will effect the entrainment of the percentage of air shown in the following tabulation in the concrete as discharged from the mixer:

Coarse aggregate, maximum size in inches:	*Total air, percent by volume of concrete*
¾	6±1
1½	5±1
3	4½±1

The agent in solution shall be maintained at a uniform strength and shall be added to the batch in a portion of the mixing water. This solution shall be batched by means of a mechanical batcher capable of accurate measurement. When calcium chloride is being used in the concrete, the portion of the mixing water containing the air-entraining agent shall be introduced separately into the mixer.

G–48. *Water.*—The water used in concrete, mortar, and grout, shall be free from objectionable quantities of silt, organic matter, alkali, salts, and other impurities.

G–49. *Sand.*—(a) *General.*—The term "sand" is used to designate aggregate in which the maximum size of particles is three-sixteenths of an inch. Sand for concrete, mortar, and grout shall be natural sand, except that crushed sand may be used to make up deficiencies in the natural sand grading. Sand, as delivered to the batching plant, shall have a uniform and stable moisture content.

(b) *Quality.*—The sand shall consist of hard, dense, durable, uncoated rock fragments. The maximum percentages of deleterious substances in the sand, as delivered to the mixer, shall not exceed the following values:

	Percent by weight
Material passing No. 200 screen (designation 16)	3
Shale (designation 17)	1
Coal (designation 17)	1
Clay lumps (designation 13)	1
Total of other deleterious substances (such as alkali, mica, coated grains, soft flaky particles, and loam)	2

The sum of the percentages of all deleterious substances shall not exceed 5 percent, by weight. Sand producing a color darker than the standard in the colorimetric test for organic impurities (designation 14) may be rejected. Sand having a specific gravity (designation 9, saturated surface-dry basis) of less than 2.60 may be rejected. The sand may be rejected if the portion retained on a No. 50 screen, when subjected to five cycles of the sodium sulfate test for soundness (designation 19), shows a weighted average loss of more than 8 percent, by weight.

(c) *Grading.*—The sand as batched shall be well graded, and when tested by means of standard screens (designation 4), shall conform to the following limits:

Screen No.:	*Individual percent, by weight, retained on screen*
4	0 to 5.
8	5 to 15.[1]
16	10 to 25.[1]
30	10 to 30.
50	15 to 35.
100	12 to 20.
Pan	3 to 7.

[1] If the individual percent retained on the No. 16 screen is 20 percent or less, the maximum limit for the individual percent retained on the No. 8 screen may be increased to 20 percent.

G–50. *Coarse Aggregate.*—(a) *General.*—The term "coarse aggregate" is used to designate aggregate graded from 3/16 inch to 3 inches, or any size or range of sizes within such limits. The coarse aggregate shall be reasonably well graded within the nominal size ranges hereinafter specified. Coarse aggregate for concrete shall consist of natural gravel or crushed rock, or a mixture of natural gravel and crushed rock. Coarse aggre-

gate, as delivered to the batching plant, shall have a uniform and stable moisture content.

(b) *Quality.*—The coarse aggregate shall consist of hard, dense, durable, uncoated rock fragments. The percentages of deleterious substances in any size of coarse aggregate, as delivered to the mixer, shall not exceed the following values:

	Percent, by weight
Material passing No. 200 screen (designation 16)	1
Shale (designation 18)	1
Coal (designation 18)	1
Clay lumps (designation 13)	½
Other deleterious substances	1

The sum of the percentages of all deleterious substances in any size, as delivered to the mixer, shall not exceed 3 percent, by weight. Coarse aggregate may be rejected if it fails to meet the following test requirements:

Los Angeles rattler test (designation 21).—If the loss, using grading A, exceeds 10 percent, by weight, at 100 revolutions or 40 percent, by weight, at 500 revolutions.

Sodium sulfate test for soundness (designation 19).—If the weighted average loss, after five cycles, is more than 10 percent, by weight.

Specific gravity (designation 10).—If the specific gravity (saturated surface-dry basis), is less than 2.60.

(c) *Separation.*—The coarse aggregate shall be separated into nominal sizes and shall be graded as follows:

Designation of size, inches	Nominal size range, inches	Minimum percent retained on screens indicated
¾	$^3/_{16}$ to ¾	50 percent on ⅜ inch.
1½	¾ to 1½	25 percent on 1¼ inch.
3	1½ to 3	25 percent on 2½ inch.

Coarse aggregate shall be screened at the batching bins over stationary sloping screens having slotted openings three-sixteenths of an inch in the narrow dimension. Material passing the $^3/_{16}$-inch screens shall be wasted as directed. (NOTE.—When large quantities of concrete are involved in the work, normally in excess of 10,000 cubic yards, the specifications should, in lieu of requiring screening of the coarse aggregate over stationary sloping screens, require the coarse aggregate to be finish screened over vibrating screens mounted on the batching plant or, at the option of the contractor, mounted on the ground adjacent to the batching plant. When finish screening is specified, the specifications should also provide for the finish screens, if installed on the batching plant, to be so mounted that the vibration of the screens will not affect the accuracy of the batching scales and that the finished products, after finish screening, shall pass directly to the batching plant bins. In such cases, the percentage of material passing the undersize test screen (significant undersize) should not exceed 2 percent, by weight.)

Separation of the coarse aggregate into the specified sizes shall be such that, when the aggregate as batched, is tested by screening on the screens designated in the following tabulation, the material passing the undersize test screen (significant undersize) shall not exceed 3 percent by weight, and all material shall pass the oversize test screen:

Size of aggregate, inches	Size of square opening in screen	
	For undersize test	For oversize test
¾	No. 5 mesh (United States standard screen)	⅞ inch
1½	⅝ inch	1¾ inches.
3	1¼ inches	3½ inches.

Screens used in making the tests for undersize and oversize shall conform to the requirements of ASTM Designation E 11–39, with respect to permissible variations in average openings. (NOTE.—If the concrete work is such that use of 3-inch aggregate is not desirable, the requirements contained in this paragraph for 3-inch aggregate should be deleted.)

G–51. *Batching.*—(a) *General.*—The contractor shall provide equipment and shall maintain and operate the equipment as required to accurately determine and control the amount of each separate ingredient entering the concrete. The amounts of bulk cement, sand, and each size of coarse aggregate entering each batch of concrete shall be determined by weighing, and the amount of water shall be determined by weighing or volumetric measurement. Where sacked cement is used, the concrete shall be proportioned on the basis of integral sacks of cement unless the cement is weighed.

When bulk cement and aggregates are hauled from a central batching plant to the mixers, the cement for each batch shall either be placed in an

individual compartment which, during transit, will prevent the cement from intermingling with the aggregates and will prevent loss of cement, or be completely enfolded in and covered by the aggregates by loading the cement and aggregates for each batch simultaneously into the batch compartment. Each batch compartment shall be of sufficient capacity to prevent loss in transit and to prevent spilling and intermingling of batches as compartments are being emptied. If the cement is enfolded in aggregates containing moisture, and delays occur between filling and emptying the compartments, the contractor shall, at his own expense, add extra cement to each batch in accordance with the following schedule:

Hours of contact between cement and wet aggregates:	*Additional cement required, percent*
0 to 2	0
2 to 3	5
3 to 4	10
4 to 5	15
5 to 6	20
Over 6	[1]

[1] Batch will be rejected.

(b) *Equipment.*—The weighing equipment shall conform to the applicable requirements of Federal Specification AAA–S–121b for such equipment, except that accuracy to within 0.4 percent of the net load being weighed will be satisfactory, and the equipment shall be capable of ready adjustment for compensating for the varying weight of any moisture contained in the aggregates and for effecting changes in concrete mix proportions. Batching equipment shall be constructed and operated so that the combined inaccuracies in feeding and measuring the materials will not exceed 1½ percent for water or weighed cement and 2 percent for each size of aggregate.

Bulk cement shall be weighed in an individual hopper and shall be kept separate from the aggregates until the batch ingredients are discharged from the batching hopper. The cement hopper may be attached to a separate scale for individual weighing, or may be attached to the aggregate scale for cumulative weighing. If materials are weighed cumulatively by dial scales, the cement shall be weighed before the other ingredients. If the materials are weighed cumulatively by beam scales, a separate beam shall be provided for weighing each material. Weighing equipment and the water measuring device shall be in full view of the operator.

The contractor shall provide standard test weights and any other auxiliary equipment required for checking the operating performance of each scale or other measuring device and shall make periodic tests over the ranges of measurements involved in the batching operations. The tests shall be made in the presence of a representative of the contracting authority, and shall be adequate to prove the accuracy of the measuring devices. Unless otherwise directed, tests of equipment in operation shall be made at least once every month. The contractor shall make such adjustments, repairs, or replacements as may be necessary to meet the specified requirements for accuracy of measurement.

The operating mechanism in the water-measuring device shall be such that leakage will not occur when the valves are closed. Water tanks on portable mixers shall be constructed so that the indicating device will register, within the specified limit of accuracy, the quantity of water discharged, regardless of the inclination of the mixer setting.

G–52. *Mixing.*—The concrete ingredients shall be mixed in a batch mixer for not less than 1½ minutes after all the ingredients, except the full amount of water, are in the mixer: *Provided*, That the mixing time may be reduced to 1¼ minutes if, when determined in accordance with the provisions of designation 26, the unit weight of air-free mortar in samples taken from the first and last portions of the batch as discharged from the mixer does not vary more than 0.8 percent from the average of the two mortar weights, the average variability for six batches does not exceed 0.5 percent, and the weight of coarse aggregate per cubic foot does not vary more than 5.0 percent from the average of the two weights of coarse aggregates. The contracting authority reserves the right to increase the mixing time when the charging and mixing operations fail to produce a concrete batch throughout which the ingredients are uniformly distributed and the consistency is uniform. The concrete, as discharged from the mixer, shall be uniform in composition and consistency throughout the mixed batch, and from batch to batch except where changes in composition or consistency are required.

Water shall be added prior to, during, and fol-

lowing the mixer-charging operations. Excessive overmixing requiring addition of water to preserve the required concrete consistency will not be permitted. Truck mixers will be permitted only when the mixers and their operation are such that the concrete throughout the mixed batch and from batch to batch is uniform with respect to consistency and grading. Any concrete retained in truck mixers so long as to require additional water to permit satisfactory placing shall be wasted. Any mixer that at any time produces unsatisfactory results, shall be repaired promptly and effectively or shall be replaced. Mixers in centralized batching and mixing plants shall be arranged so that mixing action in the mixers can be observed from a location convenient to the mixing plant operator's station. Mixers shall not be loaded in excess of their rated capacity unless specifically authorized. Each mixer shall be equipped with a mechanically operated timing and signaling device which will indicate and assure the completion of the required mixing period and will count the batches.

G–53. *Temperature of Concrete.* The temperature of concrete, when it is being placed, shall be not more than 90° F., and not less than 40° F. in moderate weather or 50° F. in weather during which the mean daily temperature drops below 40° F. Concrete ingredients shall not be heated to a temperature higher than that necessary to keep the temperature of the mixed concrete, as placed, from falling below the specified 50° F. Methods of heating concrete ingredients shall be subject to approval.

When the temperature of the concrete, as placed, may be between 80° F. and 90° F., the concrete shall be mixed at the job site and discharged into the work immediately after mixing. If concrete is placed when the weather is such that the temperature of the concrete will exceed 90° F., as determined by the contracting authority, the contractor shall employ effective means, such as precooling of aggregates and mixing water and placing at night, as necessary to maintain the temperature of the concrete, as it is placed, below 90° F. The contractor shall be entitled to no additional compensation on account of the foregoing requirements.

G–54. *Forms.* (a) *General.* Forms shall be used, wherever necessary, to confine the concrete and shape it to the required lines. Forms shall have sufficient strength to withstand the pressure resulting from placement and vibration of the concrete, and shall be maintained rigidly in position. Forms shall be sufficiently tight to prevent loss of mortar from the concrete. Chamfer strips shall be placed in the corners of forms so as to produce beveled edges on permanently exposed concrete surfaces. Interior angles on such surfaces and edges at formed joints will not require beveling unless requirement for beveling is indicated on the drawings.

(b) *Form Sheathing and Lining.* Wood sheathing or lining shall be of such kind and quality, or shall be so treated or coated, that there will be no chemical deterioration or discoloration of the formed concrete surfaces. Where pine is used for form sheathing, the lumber shall be pinus ponderosa in accordance with the Standard Grading Rules of the Western Pine Association or shall be other lumber of a grading equivalent to that specified for pine. Plywood used for form sheathing or lining shall be grade B–B, interior, or better, as described in the commercial standards of the Douglas-Fir Plywood Association. Form sheathing or lining shall be of wood or steel conforming to the following requirements, or may be of other materials producing equivalent results:

Required finish of formed surface	Wood sheathing or lining	Steel sheathing or lining [1]
F1	Any grade—S2E	Steel sheathing or steel lining permitted
F2	No. 2 common or better, pine shiplap, or plywood sheathing or lining.	Steel sheathing permitted. Steel lining permitted if approved

[1] Steel "sheathing" denotes steel sheets not supported by a backing of wood boards. Steel "lining" denotes thin steel sheets supported by a backing of wood boards.

The type and condition of form sheathing and lining, and the fabrication of forms for finish F2 shall be such that the form surfaces will be even and uniform. The ability of forms to withstand distortion caused by placement and vibration of concrete shall be such that formed surfaces will conform with applicable requirements of these specifications pertaining to finish of formed surfaces.

(c) *Form Ties.* Embedded ties for holding forms shall remain embedded and, except where F1 finish is permitted, shall terminate not less

than two diameters or twice the minimum dimension of the tie in the clear of the formed faces of the concrete. Where F1 finish is permitted, ties may be cut off flush with the formed surfaces. The ties shall be constructed so that removal of the ends or end fasteners can be accomplished without causing appreciable spalling at the faces of the concrete. Recesses resulting from removal of the ends of form ties shall be filled in accordance with the provisions for repair of concrete.

(d) *Cleaning and Oiling of Forms.*—At the time the concrete is placed in the forms, the surfaces of the forms shall be free from encrustations of mortar, grout, or other foreign material. Before concrete is placed, the surfaces of the forms shall be oiled with a commercial form oil that will effectively prevent sticking and will not stain the concrete surfaces. For wood forms, form oil shall consist of straight, refined, pale, paraffin mineral oil. For steel forms, form oil shall consist of refined mineral oil suitably compounded with one or more ingredients which are appropriate for the purpose.

(e) *Removal of Forms.*—To facilitate satisfactory progress with the specified curing, and enable earliest practicable repair of surface imperfections, forms shall be removed as soon as the concrete has hardened sufficiently to prevent damage by careful form removal. Forms on upper sloping faces of concrete, such as forms on the water sides of warped transitions, shall be removed as soon as the concrete has attained sufficient stiffness to prevent sagging. Any needed repairs or treatment required on such sloping surfaces shall be performed at once, and be followed immediately by the specified curing.

To avoid excessive stresses in the concrete that might result from swelling of the forms, wood forms for wall openings shall be loosened as soon as this can be accomplished without damage to the concrete. Forms for the openings shall be constructed so as to facilitate such loosening. Forms for conduits and tunnel lining shall not be removed until the strength of the concrete is such that form removal will not result in perceptible cracking, spalling, or breaking of edges or surfaces, or other damage to the concrete. Forms shall be removed with care so as to avoid injury to the concrete, and any concrete so damaged shall be repaired in accordance with the provisions for repair of concrete.

G–55. *Reinforcement Bars *(and Fabric).*—(a) *Furnishing Reinforcement.*—The contractor shall furnish all the reinforcement required for completion of the work. Reinforcement bars shall conform to Federal Specification QQ–S–632, type II, grade C through grade G: *Provided*, That all bars in structures required to be bent to a radius of bend of 50 diameters or less shall be intermediate-grade billet steel (grade C). *(Reinforcement fabric shall be a standard type of electrically welded wire fabric conforming to the requirements of ASTM Designation A 185.)

(b) *Placing Reinforcement.*—Steel reinforcement shall be placed in the concrete wherever shown on the drawings, or where directed. Unless otherwise shown on the drawings, or directed, measurements made in placing the bars shall be to the centerlines of the bars. Reinforcement will be inspected for compliance with requirements as to size, shape, length, splicing, position, and amount after it has been placed.

Before reinforcement is placed, the surfaces of the reinforcement and the surfaces of any metal supports shall be cleaned of heavy flaky rust, loose mill scale, dirt, grease, or other foreign substances which, in the opinion of the contracting authority, are objectionable. Heavy flaky rust can be removed by firm rubbing with burlap or equivalent treatment, if considered objectionable. After being placed, the reinforcement shall be maintained in a clean condition until it is completely embedded in the concrete.

Reinforcement shall be accurately placed and secured in position so that it will not be displaced during placing of the concrete, and special care shall be exercised to prevent any disturbance of the reinforcement in concrete that has already been placed. Metal chairs, metal hangers, metal spacers, or other satisfactory metal supports may be furnished and used by the contractor for supporting reinforcement. The reinforcement in structures shall be so placed that there will be a clear distance of at least 1 inch between the reinforcement and any anchor bolts or other embedded metalwork.

G–56. *Tolerances for Concrete Construction.*—Permissible surface irregularities for the various classes of concrete surface finish are specified in section G–59 (Finishes and Finishing); they are

*Revise or delete as appropriate.

Tolerances in Dams and Appurtenant Works

ALL STRUCTURES		
Variation of the constructed linear outline from established position in plan.	In 20 feet In 40 feet	½ inch ¾ inch
Variation of dimensions to individual structure features from established positions.	In 80 feet or more In buried construction, twice the above amounts.	1¼ inches
Variation from the plumb, from the specified batter, or from the curved surfaces of all structures, including the lines and surfaces of columns, walls, piers, buttresses, arch sections, vertical joint grooves, and visible arrises.	In 10 feet In 20 feet In 40 feet or more In buried construction, twice the above amounts.	½ inch ¾ inch 1¼ inches
Variation from the level or from the grades indicated on the drawings in slabs, beams, soffits, horizontal joint grooves and visible arrises.	In 10 feet In 30 feet or more In buried construction, twice the above amounts.	¼ inch ½ inch
Variation in cross-sectional dimensions of columns, beams, buttresses, piers, and similar members.	Minus Plus	¼ inch ½ inch
Variation in the thickness of slabs, walls, arch sections, and similar members.	Minus Plus	¼ inch ½ inch.
FOOTINGS FOR COLUMNS, PIERS, WALLS, BUTTRESSES, AND SIMILAR MEMBERS		
Variation of dimensions in plan	Minus Plus	½ inch. 2 inches.
Misplacement of eccentricity	2 percent of footing width in the direction of misplacement but not more than 2 inches.	
Reduction in thickness	5 percent of specified thickness.	
SILLS AND SIDE WALLS FOR RADIAL GATES AND SIMILAR WATERTIGHT JOINTS		
Variation from the plumb and level	Not greater than a rate of ⅛ inch in 10 feet.	

Tolerances for Concrete Tunnel Lining and Monolithic Conduits

Departure from established alinement or from established grade	Free-flow tunnels and conduits High-velocity tunnels and conduits Railroad tunnels	1 inch ½ inch. 1 inch
Variation in thickness, at any point	Tunnel lining Conduits Conduits	Minus 0. Minus 2½ percent or ¼ inch, whichever is greater Plus 5 percent or ½ inch, whichever is greater.
Variation from inside dimensions	½ of 1 percent.	

Tolerances for Placing Reinforcement Steel

Variation of protective covering	With 2-inch cover With 3-inch cover	¼ inch. ½ inch.
Variation from indicated spacing	1 inch	

defined as "finishes," and are to be distinguished from tolerances as described herein. The intent of this paragraph is to establish tolerances that are consistent with modern construction practice, yet governed by the effect that permissible deviations will have upon the structural action or operational function of the structure. Deviations from the established lines, grades, and dimensions will be permitted to the extent set forth herein: *Provided*, That the contracting authority reserves the right to diminish the tolerances set forth herein if such tolerances impair the structural action or operational function of a structure.

Where tolerances are not stated in the specifications or drawings for any individual structure or feature thereof, permissible deviations will be interpreted conformably to the provisions of this paragraph. Notations on the drawings of specific maximum or minimum tolerances in connection with any dimensions shall be considered as supplemental to the tolerances specified herein. The contractor shall be responsible for setting and maintaining concrete forms sufficiently within the tolerance limits so as to insure completed work within the tolerances specified herein. Concrete work that exceeds the tolerance limits specified in the following tabulation shall be remedied or removed and replaced at the expense of and by the contractor.

G-57. *Preparations for Placing.*—(a) *General.*—No concrete shall be placed until all formwork, installation of parts to be embedded, and preparation of surfaces involved in the placing have been approved. No concrete shall be placed in water except with the written permission of the contracting authority, and the method of depositing the concrete shall be subject to his approval. Concrete shall not be placed in running water and shall not be subjected to the action of running water until after the concrete has hardened. All surfaces of forms and embedded materials that have become encrusted with dried mortar or grout from concrete previously placed shall be cleaned of all such mortar or grout before the surrounding or adjacent concrete is placed.

(b) *Foundation Surfaces.*—Immediately before placing concrete, all surfaces of foundations upon or against which the concrete is to be placed, shall be free from standing water, mud, and debris. All surfaces of rock upon or against which the concrete is to be placed, shall, in addition to the foregoing requirements, be clean and free from oil, objectionable coatings, and loose, semidetached, or unsound fragments. Earth foundations shall be free from frost or ice when concrete is placed upon or against them. The surfaces of absorptive foundations against which concrete is to be placed shall be moistened thoroughly so that moisture will not be drawn from the freshly placed concrete.

(c) *Surfaces of Construction and Contraction Joints.*—Concrete surfaces upon or against which concrete is to be placed and to which new concrete is to adhere, that have become so rigid that the new concrete cannot be incorporated integrally with that previously placed, are defined as construction joints. The surfaces of construction joints shall be clean and damp when covered with fresh concrete or mortar. Cleaning shall consist of the removal of all laitance, loose or defective concrete, coatings, sand, sealing compound if used, and other foreign material.

The surfaces of construction joints shall be wet-sandblasted, and then thoroughly washed. The sandblasting and washing shall be performed at the last opportunity prior to placing of concrete. The surfaces of all construction joints shall be washed thoroughly with air-water jets immediately prior to placement of adjoining concrete. All pools of water shall be removed from the surfaces of construction joints before the new concrete is placed. The surfaces of all contraction joints shall be cleaned thoroughly of accretions of concrete or other foreign material by scraping, chipping, or other means approved by the contracting authority. (Note.—Wet sandblasting of surfaces of construction joints is usually required for these structures or portions of structures where watertight construction joints are essential. Delete references to this requirement when inapplicable, or apply the requirement to applicable features only.)

G-58. *Placing.*—(a) *Transporting.*—The methods and equipment used for transporting concrete and the time that elapses during transportation shall be such as will not cause appreciable segregation of coarse aggregate, or slump loss in excess of 1 inch, in the concrete as it is delivered into the work.

(b) *Placing.*—After the surfaces have been cleaned and dampened as specified, surfaces of rock and unformed construction joints shall be

covered, wherever practicable, with a layer of mortar approximately three-eighths of an inch thick. The mortar shall have the same proportions of water, air-entraining agent, cement, and sand as the regular concrete mixture, unless otherwise directed. The water-cement ratio of the mortar in place shall not exceed that of the concrete to be placed upon it, and the consistency of the mortar shall be suitable for placing and working in the manner hereinafter specified. The mortar shall be spread uniformly, and shall be worked thoroughly in all irregularities of the surface. Concrete shall be placed immediately upon the fresh mortar.

Retempering of concrete will not be permitted. Any concrete which has become so stiff that proper placing cannot be assured, shall be wasted. Concrete shall be deposited in all cases, as nearly as practicable, directly in its final position and shall not be caused to flow such that the lateral movement will permit or cause segregation of the coarse aggregate, mortar, or water from the concrete mass. Methods and equipment employed in depositing concrete in forms shall be such as will not result in clusters or groups of coarse aggregate particles being separated from the concrete mass, but if clusters do occur, they shall be scattered before the concrete is vibrated. A few scattered individual pieces of coarse aggregate that can be restored into the mass by vibration will not be objectionable.

Concrete in tunnel lining may be placed by pumping or any other approved method. Concrete in the invert shall not be placed by pneumatic placing equipment. The equipment used in placing the concrete, and the method of its operation, shall be such as will permit introduction of the concrete into the forms without high-velocity discharge, and resultant separation. After the concrete has been built up over the arch at the start of a placement, the end of the discharge line shall be kept well buried in the concrete during placement of the arch and sidewalls to assure complete filling. The end of the discharge line shall be marked so as to indicate the depth of burial at any time. Special care shall be taken to force concrete into all irregularities in the rock surfaces and to completely fill the tunnel arch.

Placing equipment shall be operated by experienced operators only. Cold joints in tunnel lining shall be avoided where practicable. In the event of equipment breakdown, or if for any other reason continuous placing will be interrupted, the contractor shall thoroughly consolidate the concrete at such joints to a reasonably uniform and stable slope while the concrete is plastic. The concrete at the surface of such cold joints shall be cleaned and dampened as required for construction joints before being covered with fresh mortar and concrete.

Except as intercepted by joints, all formed concrete, except concrete in tunnel lining, shall be placed in continuous approximately horizontal layers, the depths of which generally shall not exceed 20 inches. Lesser depths of layers will be required where concrete in 20-inch layers cannot be placed in accordance with the requirements of these specifications. All intersections of construction joints with concrete surfaces which will be exposed to view, shall be made straight and level or plumb.

(c) *Consolidation.*—Concrete shall be consolidated to the maximum practicable density, so that it is free from pockets of coarse aggregate and entrapped air, and closes snugly against all surfaces of forms and embedded materials. Consolidation of concrete in structures and in tunnel lining invert shall be by electric- or pneumatic-driven, immersion-type vibrators. Consolidation of concrete in the side walls and arch of tunnel lining shall be by electric- or pneumatic-driven form vibrators supplemented where practicable by immersion-type vibrators. Concrete vibrators shall be operated at speeds of at least 7,000 revolutions per minute when immersed in the concrete. Form vibrators shall be rigidly attached to the forms and shall operate at speeds of at least 8,000 revolutions per minute when vibrating concrete. In consolidating each layer of concrete, the vibrator shall be operated in a near vertical position and the vibrating head shall be allowed to penetrate and revibrate the concrete in the upper portion of the underlying layer. Layers of concrete shall not be placed until the layers previously placed have been worked thoroughly as specified. Care shall be exercised to avoid contact of the vibrating head with surfaces of the forms.

G-59. *Finishes and Finishing.* (a) *General.*—Allowable deviations from plumb or level and from the alinement, profile grades, and dimensions shown on the drawings, as specified for tolerances

for concrete construction, are defined as "tolerances," and are to be distinguished from irregularities in finish as described herein. The classes of finish and the requirements for finishing of concrete surfaces shall be as herein specified or as indcated on the drawings. Finishing of concrete surfaces shall be performed only by skilled workmen. Concrete surfaces will be tested by a representative of the contracting authority, where necessary to determine whether surface irregularities are within the limits hereinafter specified.

Surface irregularities are classified as "abrupt" or "gradual." Offsets caused by displaced or misplaced form sheathing or lining or form sections, or by loose knots in forms or otherwise defective form lumber, will be considered as abrupt irregularities, and will be tested by direct measurements. All other irregularities will be considered as gradual irregularities, and will be tested by use of a template consisting of a straightedge or the equivalent thereof for curved surfaces. The length of the template will be 5 feet for testing of formed surfaces and 10 feet for testing of unformed surfaces.

(b) *Formed Surfaces.*—The classes of finish for formed concrete surfaces are designated by use of symbols F1 and F2. No sack rubbing or sandblasting will be required on formed surfaces. No grinding will be required on formed surfaces, other than that necessary for repair of surface imperfections. Unless otherwise specified or indicated on the drawings, the classes of finish shall apply as follows:

F1.—Finish F1 applies to formed surfaces upon or against which fill material or concrete is to be placed, and to that portion of the upstream face of a concrete dam that will be covered by water during the greater part of the life of the dam. The surfaces require no treatment after form removal except for repair of defective concrete and filling of holes left by the removal of fasteners from the ends of tie rods, as required in section G–60 (Repair of Concrete), and the specified curing. Correction of surface irregularities will be required for depressions only, and only for those which exceed 1 inch.

F2.—Finish F2 applies to all formed surfaces not permanently concealed by fill material or concrete, or not required to receive finish F1, such as inside surfaces of tunnel linings; many of the structures that are appurtenant to earthfill dams, including surfaces of outlet works and open spillways; galleries and tunnels in dams; and concrete dams. Surface irregularities shall not exceed one-fourth of an inch for abrupt irregularities and one-half of an inch for gradual irregularities. (NOTE.—If the work involved includes any surfaces of structures which are considered to be of special importance, such as those that will be prominently exposed to public inspection, or surfaces for which accurate alinement and evenness of surface are considered of paramount importance from the standpoint of eliminating destructive effects of water action, the allowable irregularities should be reduced [2]. Also, when velocities downstream from the gates of an outlet works will be in excess of 40 feet per second, abrupt irregularities on inside formed surfaces downstream from the gates which are not parallel to the direction of flow and offset into the flow should, for a distance of approximately 15 feet downstream from the gates, be completely eliminated by grinding on a 1 to 20 ratio of height to length, and all other abrupt irregularities should be reduced so that they do not exceed one-quarter of an inch for irregularities parallel to the direction of flow and one-eighth of an inch for irregularities not parallel to the direction of flow.)

(c) *Unformed Surfaces.*—The classes of finish for unformed concrete surfaces are designated by the symbols U1 and U2. Interior surfaces shall be sloped for drainage where shown on the drawings or directed. Surfaces which will be exposed to the weather and which would normally be level, shall be sloped for drainage. Unless the use of other slopes or level surfaces is indicated on the drawings or directed, narrow surfaces, such as tops of walls, shall be sloped approximately three-eighths of an inch per foot of width; broader surfaces, such as platforms and decks, shall be sloped approximately one-fourth of an inch per foot. Unless otherwise specified or indicated on the drawings, these classes of finish shall apply as follows:

U1.—Finish U1 (screeded finish) applies to unformed surfaces that will be covered by fill material or by concrete. Finish U1 is also used as the first stage of finish U2. Finish-

ing operations shall consist of sufficient leveling and screeding to produce even, uniform surfaces. Surface irregularities shall not exceed three-eighths of an inch.

U2.—Finish U2 (floated finish) applies to unformed surfaces not permanently concealed by fill material or concrete, such as the inverts of tunnels; floors of spillways, outlet works, and stilling basins; floors of service tunnels, and temporary diversion conduits; and tops of walls. Floating may be performed by use of hand or power-driven equipment. Floating shall be started as soon as the screeded surface has stiffened sufficiently, and shall be the minimum necessary to produce a surface that is free from screed marks and is uniform in texture. Surface irregularities shall not exceed one-fourth of an inch. Joints and edges shall be tooled where shown on the drawings, or directed. (NOTE.—If steel troweling of a surface is necessary, finishes U1 and U2 will be performed as the first stages for the steel-troweled finish, the steel troweling to be performed immediately after the floated surface has hardened sufficiently to prevent excess of fine material from being drawn to the surface; the troweling to be performed with firm pressure, such as will flatten the sandy texture of the floated surface and produce a dense, uniform surface free from blemishes and trowel marks [2]. When velocities downstream from the gates of an outlet works will be in excess of 40 feet per second, irregularities on inside unformed surfaces for a distance of approximately 15 feet downstream from the gates that are not parallel to the direction of flow and are offset into the flow, such as may occur at construction joints, or elsewhere, should be completely eliminated by grinding on a 1 to 20 ratio of height to length).

G-60. *Repair of Concrete.*—Repair of concrete shall be performed by skilled workmen. The contractor shall correct all imperfections on the concrete surfaces as necessary to produce surfaces that conform to the requirements specified for finishes and finishing. Unless otherwise approved, repair of imperfections in formed concrete shall be completed within 24 hours after removal of forms. Fins and encrustations shall be neatly removed from surfaces for which finish F2 is specified, and encrustations shall be removed from surfaces for which finish U2 is specified

Concrete that is damaged from any cause, concrete that is honeycombed, fractured, or otherwise defective, and concrete which, because of excessive surface depressions, must be excavated and built up to bring the surface to the prescribed lines, shall be removed and replaced with dry pack, mortar, or concrete, as hereinafter specified. If removal of the ends of form ties results in recesses larger than one-fourth of an inch in diameter or in the minimum dimension, the recesses shall be filled with dry pack: *Provided*, That filling of recesses in surfaces designated to receive finish F1 will be required only where the surfaces are required to be coated with dampproofing, and where the recesses are deeper than 1 inch in walls less than 12 inches thick.

Where bulges and abrupt irregularities protrude outside the limits specified for finishes and finishing on formed surfaces for which finish F2 is required, the protrusions shall be reduced by bush-hammering and grinding so that the surfaces are within the specified limits. Dry pack shall be used for filling holes having at least one surface dimension little, if any, greater than the hole depth; for narrow slots cut for repair of cracks; for grout pipe recesses; and for tie-rod fastener recesses as specified. Dry pack shall not be used for filling behind reinforcement or for filling holes that extend completely through a concrete section. Mortar filling, placed under impact by use of a mortar gun, may be used for repairing defects on surfaces designated to receive F1 and F2 finishes where the defects are too wide for dry pack filling and too shallow for concrete filling and no deeper than the far side of the reinforcement that is nearest the surface. Concrete filling shall be used for holes extending entirely through concrete sections; for holes in which no reinforcement is encountered and which are greater in area than 1 square foot, and deeper than 4 inches; and for holes in reinforced concrete which are greater in area than one-half square foot and which extend beyond reinforcement.

All materials used in the repair of concrete shall conform to the requirements of these specifications and the repairs shall be made in accordance with the procedures of the Bureau of Reclamation Concrete Manual [3]. All fillings shall be bonded tightly to the surfaces of the holes and shall be

sound and free from shrinkage cracks and drummy areas after the fillings have been cured and have dried.

G-61. *Protection.*—The contractor shall protect all concrete against injury until final acceptance by the contracting authority. Immediately following the first frost in the fall, the contractor shall be prepared to protect all concrete against freezing. After the first frost, and until the mean daily temperature in the vicinity of the work site falls below 40° F. for more than 1 day, the concrete shall be protected against freezing temperatures for not less than 48 hours after it is placed. After the mean daily temperature in the vicinity of the worksite falls below 40° F. for more than 1 day, the concrete shall be maintained at a temperature not lower than 50° F. for at least 72 hours after it is placed.

Concrete cured by membrane curing will require no additional protection from freezing if the protection at 50° F. for 72 hours is obtained by means of approved insulation in contact with the forms or concrete surfaces; otherwise, the concrete shall be protected against freezing temperatures for 72 hours immediately following the 72 hours of protection at 50° F. Concrete cured by water curing shall be protected against freezing temperatures for 3 days immediately following the 72 hours of protection at 50° F.

Discontinuance of protection against freezing temperatures shall be such that the drop in temperature of any portion of the concrete will be gradual and will not exceed 40° F. in 24 hours. After March 15, when the mean daily temperature rises above 40° F. for more than three successive days, the specified 72-hour protection at a temperature not lower than 50° F. may be discontinued for as long as the mean daily temperature remains above 40° F.: *Provided*, That the concrete shall be protected against freezing temperatures for not less than 48 hours after placement. Where artificial heat is employed, special care shall be taken to prevent the concrete from drying.

G-62 *Curing.*—(a) *General.*—Concrete shall be cured either by water curing in accordance with subsection (b), or by membrane curing in accordance with subsection (c), except as otherwise hereinafter provided. The unformed top surfaces of walls and piers shall be moistened by covering with water-saturated material or by other effective means as soon as the concrete has hardened sufficiently to prevent damage by water. These surfaces and steeply sloping and vertical-formed surfaces shall be kept completely and continually moist, prior to and during form removal, by water applied on the unformed top surfaces and allowed to pass down between the forms and the formed concrete faces. This procedure shall be followed by the specified water curing or membrane curing.

(b) *Water Curing.*—Concrete cured with water shall be kept wet for at least 14 days immediately following placement of the concrete or until covered with fresh concrete: *Provided*, That water curing of concrete may be reduced to 6 days during periods when the mean daily temperature in the vicinity of the work site is less than 40° F.: *Provided further*, That during the prescribed period of water curing when temperatures are such that concrete surfaces may freeze, water curing shall be temporarily discontinued. The concrete shall be kept wet by covering with water-saturated material or by a system of perforated pipes, mechanical sprinklers, or porous hose, or by any other approved method which will keep all surfaces to be cured continuously (not periodically) wet. Water used for curing shall meet the requirements of these specifications for water used for mixing concrete.

(c) *Membrane Curing.*—Membrane curing shall be by application of a sealing compound which forms a water-retaining membrane on the surfaces of the concrete. The sealing compound shall be white-pigmented and shall conform to "Tentative Specifications for Liquid Membrane Forming Compounds for Curing Concrete," ASTM Designation C 309–58. The compound shall be of uniform consistency and quality within each container and from shipment to shipment.

Sealing compounds shall be applied to the concrete surfaces by spraying in one coat to provide a continuous, uniform membrane over all areas. Coverage shall not exceed 150 square feet per gallon, and on rough surfaces coverage shall be decreased as necessary to obtain the required continuous membrane. The repair of surface imperfections shall not be made until after application of sealing compound.

When sealing compound is used on unformed concrete surfaces, application of the compound shall commence immediately after finishing operations are completed. When sealing compound is to be used on formed concrete surfaces, the sur-

faces shall be moistened with a light spray of water immediately after the forms are removed and shall be kept wet until the surfaces will not absorb more moisture. As soon as the surface film of moisture disappears, but while the surface still has a damp appearance, the sealing compound shall be applied. Special care shall be taken to insure ample coverage with the compound at edges, corners, and rough spots of formed surfaces. After application of the sealing compound has been completed, and the coating is dry to touch, any required repair of concrete surfaces shall be performed. Each repair, after being finished, shall be moistened and coated with sealing compound in accordance with the foregoing requirements.

Equipment for applying sealing compound and the method of application, shall be in accordance with the provisions of the Bureau of Reclamation Concrete Manual [2]. Traffic and other operations by the contractor shall be such as to avoid damage to coatings of sealing compound for a period of not less than 28 days. Where it is impossible, because of construction operations, to avoid traffic over surfaces coated with sealing compound, the membrane shall be protected by a covering of sand or earth not less than 1 inch in thickness or by other effective means. The protective covering shall not be placed until the sealing membrane is completely dry. Before final acceptance of the work, the contractor shall remove all sand or earth covering in an approved manner. Any sealing membrane that is damaged or that peels from concrete surfaces within 28 days after application, shall be repaired without delay, and in an approved manner.

Sealing compounds will be accepted on manufacturer's certification of compliance with specifications, but permission to ship on certification shall in no way relieve the contractor of the responsibility for furnishing compound meeting specifications requirements.

E. MISCELLANEOUS

G–63. *Diversion and Care of River During Construction and Removal of Water from Foundations.*—(a) *General.*—The contractor shall construct and maintain all necessary cofferdams, channels, flumes, drains, sumps, and/or other temporary diversion and protective works; shall furnish all materials required therefor; and shall furnish, install, maintain, and operate all necessary pumping and other equipment for removal of water from the various parts of the work and for maintaining the foundations and other parts of the work free from water. After having served their purpose, all cofferdams or other protective works downstream from the dam shall be removed from the river channel, or leveled to give a sightly appearance, so as not to interfere in any way with the operation or usefulness of the reservoir, and in a manner approved by the contracting authority. All cofferdams or other protective works constructed upstream from the dam and not a part of the permanent dam embankment shall be removed or leveled and graded to the extent required to prevent obstruction in any degree whatever of the flow of water to the spillway or outlet works. The contractor shall be responsible for and shall repair at his expense any damage to the foundations, structures, or any other part of the work caused by floods, water, or failure of any part of the diversion or protective works.

(b) *Plan for Diversion and Care of River During Construction.*—The contractor's plan for the diversion and care of the river during construction shall be subject to approval. The plan may be placed in operation upon approval, but nothing in this section shall relieve the contractor from full responsibility for the adequacy of the diversion and protective works. The hydrographs of ________ ________ and discharge curves for the spillway and outlet works *(and for diversion) are shown on the drawings solely for the information of the contractor in timing his construction operations to prepare for such flood storage and/or to bypass such flow as may be necessary. The contracting authority does not guarantee the reliability or accuracy of any of these curves and assumes no responsibility for any deductions, interpretations, or conclusions which may be made from the curves.

Except as otherwise provided below, the contractor shall not interrupt nor interfere with the

484556 O—60——39

natural flow of ____________________ through the damsite for any purpose without the written approval of the contracting authority. *(The contractor shall at all times, except during the closure period, pass the full natural flow of the stream through the damsite to meet prior rights for ____________________ purposes.) *(Final closure shall be made between the following dates: ____________________)

(c) *Removal of Water from Foundations.*—The contractor's method of removal of water from foundation excavations shall be subject to the approval of the contracting authority. Where excavation for cutoff trenches in embankment foundations extends below the water table in common material, the portion below the water table shall be dewatered in advance of excavation. The dewatering shall be accomplished in a manner that will prevent loss of fines from the foundation, will maintain stability of the excavated slopes and bottom of the cutoff trench, and will result in all construction operations being performed in the dry. The use of a sufficient number of properly screened wells or other equivalent methods will be approved for dewatering. The contractor will also be required to control seepage along the bottom of the cutoff trench, which may require supplementing the approved dewatering systems by pipe drains leading to sumps from which the water shall be pumped. Such pipe drains shall be of uniform diameter for each run, shall be provided with grout connections and returns at 50-foot intervals, and shall be embedded in reasonably well-graded gravel or like material.

During the placing and compacting of the embankment material in a cutoff trench, the water level at every point in the cutoff trench shall be maintained below the bottom of the embankment until the compacted embankment in the cutoff trench at that point has reached a depth of 10 feet, after which the water level shall be maintained at least 5 feet below the top of the compacted embankment. When the embankment has been constructed to an elevation which will permit the dewatering systems to maintain the water level at or below the designated elevations, as determined by the contracting authority, the pipe drains including surrounding gravel shall be filled with grout composed of water and cement or clay.

(d) *Cost.*—The cost of furnishing all labor, equipment, and materials for constructing cofferdams, dikes, channels, flumes, and other diversion and protective works; removing or leveling such works, where required; diverting the river; making required closures; maintaining the work free from water as required; disposing of materials in cofferdams; and all other work required by this section shall be included in the prices bid in the schedule for items of construction work.

G–64. *Concrete or Cement-Bound Curtain.*—The contractor shall construct a mixed-in-place cement-bound curtain, or a concrete or cement-bound curtain constructed by other methods that will be equivalent to a mixed-in-place cement-bound curtain. At least 30 days before beginning any work on the concrete or cement-bound curtain, the contractor shall submit, for approval, plans and specifications for the work. The contractor shall furnish all materials and equipment required to construct the mixed-in-place or other type concrete or cement-bound curtain. The curtain shall have a minimum effective thickness of 12 inches and shall be constructed to the depths and elevations shown on the drawings or established by the contracting authority and as close as practicable to the established lines. The cement used in the curtain shall be portland cement conforming to the requirements of section G–46 (Cement). Particular efforts should be directed toward maintaining a continuous, unbroken cutoff, by bonding around the end of the previously placed "pile" whenever shutdown periods occur and construction is again resumed. The curtain, when completed, shall provide a reasonably impervious barrier to the passage of subsurface flows. Any royalties due because of the mixed-in-place cement-bound curtain or other type concrete or cement-bound curtain being placed by patented methods or use of patented materials shall be paid by the contractor.

Measurement, for payment, of mixed-in-place cement-bound curtain or other type concrete or cement-bound curtain will be made of the area of curtain placed, based on the depth and length of the curtain along the centerline of the curtain as shown on the drawings or established by the con-

*Delete or revise as appropriate and add pertinent information on the following:
(1) Downstream requirements.
(2) What contractor will be permitted and will not be permitted to do.
(3) Work to be accomplished before permanent construction may be used for diversion purposes.
(4) Work to be accomplished before final closure can be made.

tracting authority. Payment for constructing concrete or cement-bound curtain will be made at the unit price per square yard bid therefor in the schedule, which unit price shall include the cost of all labor, materials and equipment required to complete the cutoff curtain, regardless of the type of curtain constructed: *Provided*, That payment for furnishing and handling cement will be made at the unit price per barrel bid therefor in the schedule.

G-65. *Timber Cutoff Wall.* (a) *General.*—The timber cutoff wall shall be constructed in the trench for the dam in accordance with the details shown on the drawings. The contractor shall furnish all materials for the timber cutoff wall, including all lumber, treatment materials, nails, bolts, nuts, and washers. All timber, except redwood, shall be treated in accordance with subsection (2), below. The cutoff wall shall be bedded in a firm foundation and shall be held vertically while the compacted impervious backfill is placed about it in the cutoff trench.

(b) *Materials.*—

(1) *Lumber.*—(Federal Specification MM-L-751c.) The lumber shall be of one or more of the species and of not less than the minimum requirements as specified below:

Douglas-fir.—Douglas-fir shall be "Construction" grade timbers for the planks and "Selected Framing and Timbers" for other members, both in accordance with the Standard Grading and Dressing Rules No. 15 of the West Coast Lumber Inspection Bureau.

Southern and longleaf yellow pine.—Planks shall be "square edge and sound utility timbers and heavy joists" and other timber shall be structural grades "No. 1 Structural" for longleaf yellow pine and "Dense No. 1 Structural" for southern yellow pine in accordance with the Standard Grading Rules of the Southern Pine Inspection Bureau.

Redwood.—Planks shall be "sap common dimension, joists, and timbers, 3 inches and thicker" and other timber shall be "Dense Structural" in accordance with the Standard Specifications of the California Redwood Association.

(2) *Preservative treatment.*—Preservative shall be coal-tar creosote or shall be a pentachlorophenol solution consisting of petroleum oil solvent and not less than 5 percent by weight of pentachlorophenol. Coal-tar creosote shall conform to Federal Specification TT-W-556b. Pentachlorophenol shall conform to Federal Specification TT-W-570a. The petroleum oil solvent for use with the pentachlorophenol shall conform to No. P-9 "Standard for Petroleum used in Pentachlorophenol and Copper Naphthenate Solutions" of the American Wood Preserver's Association, except that the flash point shall not be less than 200° F. with Saybolt Universal viscosities of not more than 200 seconds at 100° F. and not more than 45 seconds at 210° F.

Treatment shall be by the empty-cell process, either with or without initial air, in accordance with the "Standard Specification for the Preservative Treatment of Lumber and Timbers by Pressure Processes" No. C1 and C2 of the American Wood Preserver's Association and Federal Specification TT-W-571c. Lumber treated with coal-tar creosote shall have a minimum net retention of 10 pounds per cubic foot for Douglas-fir under 5 inches thick and 8 pounds per cubic foot for Douglas-fir 5 inches and over, and 8 pounds per cubic foot for all sizes of yellow pine. Lumber treated with pentachlorophenol solution shall have a minimum net retention of 10 pounds per cubic foot.

(3) *Nails and spikes.*—(Federal Specification FF-S-606.) Nails and spikes shall be round wire, flat head, diamond point, smooth finish, and bright.

(4) *Bolts.*—(Federal Specification FF-B-571a.) Threads for all bolts shall be coarse-thread series, free fit.

(5) *Washers.*—Washers shall be of standard commercial quality as approved.

(c) *Construction.*—The wall shall be constructed of 3- by 12-inch plank surfaced one side and 4- by 4-inch rough wales: *Provided*, That where timber members are to be in contact with rubber waterstops or elastic filler material, the contact surfaces shall be surfaced. Where required, members shall be cut to fit the web and outer bulb of the rubber waterstops. The members anchoring rubber waterstop shall be tightened to compress the bulb and web in an approved manner. Holes for bolts shall be bored with bits having the same diameter as the bolts. Joints between ends of adjoining planks shall be staggered and separated by a distance of not less than 12 inches. Each plank end shall be held with two sixtypenny nails

and each edge of each plank shall be fastened with sixtypenny nails spaced approximately 18 inches, and staggered on opposite edges of the plank. The ends of all nails shall be bent over and firmly embedded in the surface of the planks.

(d) *Measurement and Payment.*—Measurement, for payment, for treated timber for cutoff wall will be made of the number of feet board measure of planks and wales in place, and the commercial cross-sectional dimensions will be taken. Payment for furnishing and erecting treated timber for cutoff wall will be made at the unit price per thousand (1,000) feet board measure bid therefor in the schedule, which price shall include the cost of furnishing and installing all bolts, nails, and other materials incidental to construction of the cutoff wall.

G–66. *Steel Sheet Piling.*—Steel sheet piling shall be driven in locations as shown on the drawings or as directed by the contracting authority. The steel sheet piling including taper piles and special intersections, shall be furnished by the contractor. The piling shall be driven to the depths shown on the drawings or prescribed by the contracting authority, with the top of each pile at the elevation shown on the drawings or established by the contracting authority. Where required, the tops of piling shall be cut off to the required elevation. The piling shall be driven as close as practicable to the lines shown on the drawings or established by the contracting authority, so as to ensure perfect interlocking throughout the entire length of each pile. The method of driving shall be subject to the approval of the contracting authority.

Assembling frames of capped wooden piles or other suitable temporary guide structures furnished by the contractor shall be used, and the steel piling shall be assembled against the guides so that each pile is plumb at both edge and side. Such temporary guide structures shall be removed by the contractor when they have served their purpose. A cast-steel driving head shall be used for driving steel sheet piling. Piles ruptured in the interlock or otherwise injured in driving shall be pulled and new piles driven. Should boulders be encountered, the contractor shall make every effort to drive the piling to the required depth, either by moving or shattering the boulder or by deviations in the line of the piling. If at any time the forward edge of the steel-piling wall is found to be out of plumb, the piling already assembled and partly driven shall be driven to the required depth, and taper piles shall then be used to bring the forward edge plumb before additional piling is assembled or driven. The maximum permissible taper in a single pile shall be one-fourth inch per foot of length. Where welding of piles is required for field connections the welding shall be by approved methods.

The steel sheet piling furnished by the contractor shall be in accordance with the following requirements:

(1) *Inspections and tests.*—The steel piling shall be manufactured from steel conforming to ASTM Designation A 328-54 and shall be equal to United States Steel Standard Section MP–115 (nominal weight 22 pounds per square foot). All material furnished and all work done shall be subject to rigid inspection and no material shall be shipped until all tests, analyses, and final inspection have been made, or certified copies of reports of tests and analyses or manufacturer's guarantees shall have been accepted. If test specimens are required for analyses they shall be properly boxed and prepared for shipment.

(2) *Type and general description of piling.*—The piling shall have a continuous interlock rolled integral with the pile throughout its entire length. The interlock shall permit an angular movement between adjoining piles of not less than 10° in either direction from the centerline. In order to withstand difficult driving requirements the steel piling shall be hard and tough to resist battering and shock, and shall also be free from objectionable brittleness.

Measurement, for payment, of steel sheet piling will be made of the area remaining in place and on the basis of 22 pounds per square foot. Payment for furnishing and driving steel sheet piling will be made at the unit price per pound bid therefor in the schedule, which unit price shall include the cost of driving the piling, welding of field connections, cutting the tops, and pulling and replacing unsatisfactory piling as described in this section.

G–67. *Consolidation of Dam Foundation by Flooding.*—Consolidation of the dam foundation by flooding will be required in the embankment area as outlined on the drawings. The consolidation

of the dam foundation will be accomplished by progressively flooding separate sections of the foundation. The flooding shall be such that complete saturation of the foundation will be accomplished and a continuous water cover to a minimum depth of 12 inches shall be maintained. Before flooding is started, the excavation for the dam foundation shall be completed as provided in section G-9 (Excavation for Dam Embankment Foundation). At such time as is approved by the contracting authority, the contractor shall begin consolidation of the foundation by flooding with water. The flooding operations shall be continued until the desired penetration and saturation has been attained: *Provided*, That the contractor will not be required to provide plant capacity sufficient to supply water in excess of gallons per day: *Provided further*, That the contractor will not be required to maintain continuous flooding of any individual section to a minimum depth of 12 inches for longer than 60 days. In the event of plant failure, the maximum time limit of 60 days for any section will be extended by the same number of days as pumping is interrupted by plant failure, if the desired penetration and saturation has not been attained prior to that time. Additional pumping, if required in any section beyond the 60-day period, except where extension of the 60-day period is due to failure of plant, will be ordered in writing by the contracting authority and payment thereof will be made as extra work.

The quantity stated in the schedule for water for consolidation of dam foundation is an estimate only of the amount to be required, and the contractor shall be entitled to no additional compensation above the unit prices bid in the schedule by reason of any amount or none being required. The contractor shall be responsible for, and shall repair at his own expense, any damage to the dam foundation or any part of the work caused by excessive pumping or failure of pipelines or temporary dikes. No payment will be made for water lost due to failure of any part of the contractor's water-supply system or dikes. The consolidation of the dam foundation shall be scheduled so that as short a time as practicable will elapse between the time the consolidation is completed and the earth embankment is placed. Measurement, for payment, of water for consolidation of dam foundation will be made by metering the water near the point or points of discharge into the dam foundation. Meters furnished and installed by the contractor shall be tested for accuracy prior to use. Payment for water for consolidation of dam foundation will be made at the unit prices per million gallons bid therefor in the schedule, which unit prices shall include the cost of all water and the cost of all labor, materials, and operations required for continuously flooding the dam foundation as described in this section, including the cost of constructing, maintaining, and removing the necessary temporary dikes: *Provided*, That the cost of all plants shall be included in the unit price bid in the schedule for water for consolidation of dam foundation, less than gallons.

G–68. *Topsoil for Seeding.*—The item of the schedule for topsoil for seeding consists of loading, hauling, placing, spreading, and rolling selected topsoil material. The topsoil shall be placed on the downstream slope of the dam embankment at locations shown on the drawings or designated by the contracting authority. All operations involved in the placing, spreading, and rolling of the topsoil shall be subject to the approval of the contracting authority. Selected topsoil shall be obtained from approved stockpiles of materials from excavation for dam embankment foundation or from stripping from borrow areas, or from other approved sources. The material shall contain the most fertile loam available from approved sources and shall be free from excessive quantities of grass, roots, weeds, sticks, stones, or other objectionable materials. Areas to receive topsoil shall be brought to within 1 foot of the prescribed final cross section at all points and finished smooth and uniform before the topsoil is applied. Topsoil shall be evenly placed and spread over the graded area and compacted in two layers, each by one pass of a roller weighing not less than 50 pounds per linear inch of length of drum. Topsoil shall not be placed when the subgrade is frozen or in a condition otherwise detrimental to proper grading and seeding as determined by the contracting authority.

Measurement, for payment, for topsoil will be by volume, compacted in place within the lines shown on the drawings or as established by the contracting authority. Payment for topsoil for seeding will be made at the unit price per cubic yard bid therefor in the schedule, which unit

price shall include the cost of loading, hauling, placing, spreading, and rolling the topsoil, and shall also include the cost of excavating additional suitable topsoil material if sufficient quantity of such material is not obtained from approved stockpiles.

G–69. *Water for Seeded Areas.*—It is expected that it will be necessary to irrigate the seeded areas in preparing the seedbed and to promote germination and growth of the plants. The frequency of application and quantities of irrigation water used will be determined by the contracting authority. The contractor shall provide a temporary sprinkler irrigation system of pipelines or mobile water tanks to provide for uniform application of water over the entire seeded areas and complete control of the amount of water at all times to eliminate erosion. The contractor shall repair at his own expense any damage to the slopes or any part of the work caused by excessive or irregular application of irrigation water. The quantity stated in the schedule for water for seeded areas is an estimate only of the amount to be required, and the contractor shall be entitled to no additional compensation above the unit price bid in the schedule by reason of any amount or none being required.

Measurement, for payment, of water for seeded areas will be made by metering the delivery pipeline using meters furnished and installed by the contractor or by tank gallonage delivered through the sprinkler heads and applied to the seeded areas. Payment for water for seeded areas will be made at the unit price per thousand gallons bid therefor in the schedule, which unit price shall include the cost of all labor, materials, plant, and operations required for sprinkler irrigation of the seeded areas as described in this section, including the cost of removing the pipelines, if used.

G–70. *Seeding.*—Seeding shall consist of ground preparation, furnishing approved seed, sowing or planting, and covering the seed. Seeding shall be completed on areas shown on the drawings or as designated by the contracting authority. Seeding shall be done in the early spring unless otherwise authorized. The contractor shall seed at each seeding season that portion of the designated seeded areas which has been completed since the last seeding season, or since the start of construction for the first seeding season.

All seed shall comply with the seed laws of the State of ____________________, and shall be subject to all regulations of the U.S. Department of Agriculture. All seed shall be labeled and shall be subject to sampling and testing in accordance with the U.S. Department of Agriculture rules and regulations under the Federal Seed Act in effect on the date of invitation for bids. It shall be furnished in sealed standard containers except as otherwise permitted by the contracting authority. Duplicate signed copies of the vender's statement shall be furnished to the contracting authority, certifying that each container of seed delivered is at least equal to the specifications requirements. This certificate shall appear on or with all copies of invoices for the seed. Each lot of seed shall contain the following minimum percentage, by weight, of live pure seed. (List types of seed and percentage of each type.)

After the topsoil has been placed on the seeding areas and leveled and compacted to finished grade, it shall be brought to a friable condition by harrowing or otherwise loosening and mixing to a depth of at least 3 inches. All lumps and clods shall be thoroughly broken with an appropriate corrugated roller of the farm soil pulverizer type, or by other means as approved by the contracting authority. Seeds shall be sown in the amount shown in the following tabulation. (List types of seed and amounts to be sown in pounds per 1,000 square yards.)

All seeding shall be by drilling, except in such areas as are inaccessible to drilling equipment. If the drill is not equipped with press wheels the seeding operation shall be followed by rolling. Seed shall be uniformly drilled to a depth of one-third to three-fourths inch and, where feasible, in approximately horizontal rows along the face of slope areas. The seed combination shall be well mixed to insure uniform distribution of each type. In areas where drilling of seed is impracticable the seed shall be broadcast by an approved seeder. Where broadcasting must be resorted to, the rate of application shall be increased by 50 percent, maintaining the same proportions. The seed shall be sown in two directions, at right angles to each other, one-half in each direction. Broadcast seeding shall not be done in windy weather. A final check will be made of the total quantity of seed used, against the total area seeded. If such check shows that the contractor has not applied seed at the specified rate per 1,000 square yards, he will be

required to uniformly distribute additional seed to meet the shortage.

Immediately after seeding, the seeded area shall be uniformly covered at the rate of 300 pounds per 1,000 square yards with a mulch composed of ripe native hay, sudan grass or other approved hay or straw threshed from wheat, oats, barley, or rye. The mulch shall be anchored to the topsoil by a V-type wheel land packer or similar implement. In areas inaccessible for use of the land packer or similar implement, the mulch shall be covered thinly with topsoil or anchored by poultry netting and stakes. The contractor shall maintain the seeded areas until all work on the entire area has been completed and accepted. If the surface becomes gullied or otherwise damaged, it shall be restored to the finished grade and soil conditions and reseeded in accordance with this section.

Measurement, for payment, for seeding will be made of the number of 1,000 square yards of areas actually seeded at the direction of the contracting authority. Payment for seeding will be made at the unit price per 1,000 square yards bid in the schedule for seeding, which unit price shall include all labor, materials, and equipment for preparation of the ground, furnishing the seed and mulch, sowing or planting, mulching and maintaining the seeded slopes until acceptance by the contracting authority or the termination of the contract.

G–71. *Joints in Concrete.*—(a) *Construction Joints.*—The location of all construction joints in concrete work shall be subject to approval, and the joints shall be constructed in accordance with section G–57 (Preparation for Placing) and section G–58 (Placing). The entire cost of constructing the joints shall be included in the price bid in the schedule for concrete in which the joints are required.

(b) *Contraction Joints.*—Contraction joints of the types shown on the drawings shall be constructed at the locations shown. The joints shall be made by forming the concrete on one side of the joint and allowing it to set before concrete is placed on the other side of the joint. The surface of the concrete first placed at a contraction joint shall be coated with sealing compound before the concrete on the other side of the joint is placed. The sealing compound shall conform to "Tentative Specifications for Liquid Membrane-Forming Compounds for Curing Concrete," ASTM Designation C 309–53T.

Except as otherwise provided for furnishing and placing joint materials, the entire cost of constructing contraction joints shall be included in the prices bid in the schedule for the concrete in which the joints are required.

G–72. *Rubber Waterstop.* (a) *General.* Rubber waterstop shall be furnished and placed in the joints at the locations shown on the drawings or where directed. Rubber waterstop shall be furnished in accordance with the details shown on drawing No. _____. The contractor shall also furnish all labor and materials for making field splices in rubber waterstop. The contractor shall take suitable precautions to support and protect the waterstop during the progress of the work and shall repair or replace any damaged waterstop. All waterstop shall be stored in as cool a place as practicable, preferably at 70° F. or less. Waterstop shall not be stored in the open or where it will be exposed to the direct rays of the sun. All waterstop shall be protected from oil or grease.

(b) *Materials.*—

(1) *Rubber waterstop.*—The rubber waterstop shall be fabricated from a high-grade, tread-type compound. The basic polymer shall be natural rubber or a synthetic rubber. The material shall be compounded and cured to have the following physical characteristics:

	Natural rubber	Synthetic rubber
Tensile strength, pounds per square inch, minimum.	3,500	3,000.
Tensile strength at 300-percent modulus, pounds per square inch, minimum.	1,450	1,150.
Elongation at break, percent, minimum.	500	450.
Shore durometer (type A)	60 to 70	60 to 70
Specific gravity	1.15±0.03	1.15±0.03
Absorption of water, by weight, percent maximum (2 days at 70° C.)	5	5.
Compression set (constant deflection) percent of original deflection, maximum	30.	30.
Tensile strength after oxygen pressure test (48 hours, 70° C., 300 pounds per square inch) percent of tensile strength before aging, minimum.	80.	80.
Elongation after oxygen pressure test (48 hours, 70° C., 300 pounds per square inch) percent of elongation before aging, minimum.	80.	80.

(2) *Gum rubber and rubber cement.*—Gum rubber and rubber cement shall be suitable for making field connections in rubber waterstop as described in subsection (e)(2) below.

(c) *Fabrication.*—The rubber waterstop may be molded or extruded. All material shall be molded or extruded and cured in such a manner that any cross section will be dense, homogeneous, and free from porosity and other imperfections. Minor surface defects such as surface peel covering less than 1 square inch, surface cavities or bumps less than $\frac{1}{4}$ inch in longest lateral dimensions and less than $\frac{1}{16}$ inch deep will be acceptable. Suck back along flash lines of molded goods will be acceptable if not more than $\frac{1}{16}$ inch wide, $\frac{1}{16}$ inch deep, and not more than 2 feet long. The tolerances shown on the drawing shall govern all cross-sectional dimensions. Any defects which are not within the above limitations either shall be repaired as approved by the contracting authority or shall be removed from the finished product by cutting out a length of waterstop containing such defects and splicing the waterstop at that point. All factory splices shall be molded splices. Molded splices shall be made by vulcanizing the splices in a steel mold for a time sufficient to produce maximum strength in the splice. All molded splices shall withstand being bent 180° around a 2-inch-diameter pin without any separation at the splice.

*(Rubber waterstop for joints in the barrel or box portions of siphons, culverts, or other pressure conduits shall be furnished in continuous circular hoops. The hoops shall be fabricated from straight strip waterstop with the ends spliced together with molded splices to form a closed ring of the specified circumferential length. A tolerance of plus or minus 2 inches will be permitted in the circumferential length of each hoop.)

(d) *Inspection and Tests.*—Rubber waterstop will be subject to inspection and tests before shipment. Material for tests shall be furnished by the manufacturer and all tests shall be made at the place of the manufacturer of the rubber waterstop.

Except as otherwise provided below, general sampling procedures and definitions of terms used herein shall be in accordance with section 6 of Federal Test Method Standard No. 601.

*Revise or delete as appropriate.

(1) *Sampling for inspection.*—All materials furnished will be subject to visual inspection for defects, imperfections, and physical dimensions.

(2) *Sampling for tests.*—Samples for laboratory tests to determine physical properties of the compound shall be taken in accordance with a random process to obtain the following number of test units from each separate purchase order:

Size of purchase order, linear feet:	*Number o test unit*
500 or less	1
501 to 1,000	2
1,001 to 5,000	4
5,001 to 10,000	8
Over 10,000	15

At the option of the manufacturer, laboratory tests to determine physical properties of the rubber waterstop required to be furnished under these specifications shall be performed on test specimens cut from (1) test units taken from the finished rubber product, or (2) substitute samples furnished in accordance with paragraph 3.5 of section 6, Federal Test Method Standard No. 601.

(3) *Methods of tests.*—Tests shall be made in accordance with the following methods described in Federal Test Method Standard No. 601:

Tensile strength	Method 4111.
Elongation	Method 4121.
Durometer	Method 3021.
Specific gravity	Method 14011.
Water absorption	Method 6251.
Compression set	Method 3311.
Oxygen pressure test	Method 7111, except the time interval of test shall be $48 \pm \frac{1}{2}$ hours.

(e) *Installation.*—The waterstop shall be installed with approximately one-half of the width of the material embedded in the concrete on each side of the joint. Care shall be exercised in placing and vibrating the concrete about the waterstop to insure complete filling of the concrete forms under and about the waterstop, and to obtain a continuous bond between the concrete and the waterstop at all points around the periphery of the waterstop. In the event the waterstop is installed in the concrete on one side of a joint more than 1 month prior to the scheduled date of

placing the concrete on the other side of the joint, the exposed waterstop shall be covered or shaded to protect it from the direct rays of the sun during the exposure.

(1) *Field splices.*—All splices in continuous circular hoops shall be molded splices. Splices in waterstop other than continuous circular hoops shall be made either as molded splices or by the use of molded rubber splicing sleeves. All molded splices shall be made by vulcanizing the splices in a steel mold as follows. The adjoining ends at splices shall be beveled at an angle of 45° or flatter by the use of a saw and miter box so that the ends to be spliced together will be pressed together when the mold is closed. The beveled ends and the sides for at least one-fourth inch back from the ends shall be buffed thoroughly to provide clean, rough surfaces. All buffed surfaces shall be given two thin coats of rubber cement and each coat shall be permitted to dry thoroughly. A piece of gum rubber cut to the same dimensions as the beveled face shall then be applied to the end of one strip after removing the cloth backing from the gum rubber. The adjoining strip shall then be placed accurately in position, and all edges shall be stitched thoroughly together with a suitable hand stitcher. The mold shall be heated to a temperature of 290° F. before the splice is placed in the mold. The prepared splice shall be placed in the mold with the splice in the center of the mold, and the mold shall be closed tightly to prevent slipping during the vulcanizing process. The splice shall remain in the mold 25 minutes after the mold is closed completely, during which time the mold shall be maintained at a temperature of 290° F.

The contractor shall furnish all materials for molded splices, all field splicing molds, and electrical energy for heating the molds.

If field splices are made with molded rubber splicing sleeves, the sleeve material shall conform to the requirements of subsection (b) above. Where rubber sleeves are to be used, the contractor shall submit to the contracting authority for approval, detail drawings of the sleeves, certified materials data, and one sample of each size of sleeve he proposes to furnish. Rubber sleeves shall not be used in making field splices until approved by the contracting authority. Rubber sleeve-type field splices shall be made in accordance with the following requirements, and the method of making field splices with rubber sleeves will be subject to the approval and direction of the contracting authority. The ends of waterstop to be spliced shall be buffed smooth of any imperfections before insertion into the sleeve. The inside surface of the sleeve and the outside surface of the waterstop to be in contact after splicing shall be thoroughly coated with field splicing cement. After the waterstop has been inserted into the sleeve with the abutting ends of the waterstop centered in the sleeve, the sleeve shall be pressed tightly against the waterstop around the entire periphery to obtain a tight fit at all points and shall be held in such position by blocking and use of C-clamps until the cement has dried.

Each finished splice, either molded or sleeve spliced, shall withstand a bend test by bending the waterstop 180° around a 2-inch-diameter pin without showing any separation at the splice.

(f) *Measurement and Payment.*—Measurement, for payment, of rubber waterstop will be made of the number of linear feet of waterstop in place, measured along the centerline of the waterstop. Payment for furnishing and placing rubber waterstop will be made at the unit price per linear foot bid therefor in the schedule, which unit price shall include the cost of furnishing all material, making the splices, and installing the waterstop.

G–73. *Elastic Filler Material in Concrete Joints.* (a) *General.*—Elastic filler material consisting of sponge rubber shall be furnished and placed in the concrete joints where shown on the drawings.

(b) *Materials.*—Sponge rubber shall conform to Federal Specification HH–F–341a, for type I, class A, sponge rubber. Sponge rubber shall be stored in as cool a place as practicable, preferably at 70° F. or less, and in no case shall the rubber be stored in the open, exposed to the direct rays of the sun. Copper nails shall conform to Federal Specification FF–N–105 for common copper wire nails.

(c) *Installation.*—Copper nails shall be embedded in the first-placed concrete in such a manner that the nail points will protrude from the joint

surface to be covered with the elastic filler material. The elastic filler material shall be cut to size and shape of the joint surface and held in place by the protruding copper nails. Joints between adjoining portions of the filler material shall be sufficiently tight to prevent mortar from seeping through such joints. Unless otherwise directed, the edges of the elastic filler material shall be placed flush with the finished surface of the concrete or to the bottom edge of chamfers. Surfaces of concrete to be in contact at the joint shall be painted with sealing compound.

(d) *Measurement and Payment*[14].—Measurement, for payment, of elastic filler material will be made of the area of material in place. Payment for furnishing and placing elastic filler material in joints will be made at the unit price per square foot bid therefor in the schedule.

(e) *Cost*[14].—The entire cost of furnishing and placing elastic filler material in joints shall be included in the unit price per cubic yard bid in the schedule for concrete in which the elastic filler material is placed.

G–74. *Metal Seals.*—(a) *General.*—Metal seals of corrosion-resistant metal shall be placed in joints in the structures where shown on the drawings, and elsewhere as directed. The contractor shall furnish all materials for metal seals, including metal seals, materials for welding or brazing metal seals, and washers and nails for fastening the seals to the forms. Details of the shape and of the placing of the seals are shown on the drawings. The seals shall be jointed carefully together by welding or brazing so as to form continuous watertight diaphragms in the joints. Adequate provisions shall be made to support and protect the seals during the progress of the work. The contractor shall replace or repair any metal seals punctured or damaged before final acceptance of the work.

(b) *Materials.*—Materials for metal seals shall conform to the following specifications unless otherwise approved in writing by the contracting authority:

(1) *Metal seals.*—Metal seals shall be made from one of the following materials:

Copper.—Federal Specification QQ–C–502a, 24 ounces per square foot, 0.032 inch thick, soft temper.

Corrosion-resisting steel.—Federal Specification QQ–S–766a, class 6, 18 percent Cr, 8 percent Ni (0.10 percent C maximum). (Ti or Cb stabilized), condition A (annealed), hot- or cold-rolled finish, No. 20 gage, United States Standard (0.0375 inch thick).

(2) *Nails.*—Federal Specification FF–S–606.

(3) *Miscellaneous materials.*—Miscellaneous materials not covered herein by detailed specifications shall be of standard commercial quality, of a type and composition approved by the contracting authority.

(c) *Measurement and Payment.*—Measurement, for payment, of furnishing and placing metal seals will be made of the linear feet of seals in place, and no allowance will be made for lap at joints. Payment for furnishing and placing metal seals will be made at the unit price per linear foot bid therefore in the schedule, which unit price shall include the cost of furnishing, storing, handling, forming, welding, or brazing, and placing the metal seals; furnishing all welding or brazing rods, washers, and nails; and maintaining the metal seals free from damage, as described in this section.

G–75. *Metal Waterstops.*—(a) *General.*—Metal waterstops shall be provided at all transverse construction joints in the tunnel lining. The metal waterstops shall consist of plates of $\frac{3}{16}$- by 8-inch wrought iron bent and welded to the size and shape of the structure at the joint and coated with plastic compound. The metal plates, welding rods, and plastic compound shall be furnished by the contractor.

(b) *Placing.*—The waterstops shall be located in the concrete of the structures as shown on the drawings. The contractor shall cut, shape, and weld the plates and place them in the construction joints as herein provided. The exposed half of the metal waterstops shall be coated completely to a thickness of approximately one-sixteenth inch with plastic compound applied cold. Adequate provisions shall be made to support and protect the metal stops during the progress of the work. The contractor shall replace or repair any waterstops which are damaged before final acceptance of the work.

(c) *Materials.*—

(1) *Wrought-iron plates.*—The wrought-iron plates shall conform to ASTM Designation A 42–55.

[14] Subs. (d) and (e) are alternates.

(2) *Plastic compound.*—The plastic compound shall consist of asphalt dispersed in water by means of a mineral emulsifying agent. The compound shall be heavy-bodied, of smooth plastic consistency, and free from lumps and sediment. When applied at room temperature in one coat to a smooth metal surface at a thickness of one-sixteenth inch and the specimen is then immediately placed in a vertical position, the compound shall be tightly adherent and shall not run or sag while the coating is drying and after it has dried. The asphalt component shall comprise at least 50 percent, by weight, of the compound, shall have a softening point (ASTM Designation D 36–26) of 100° F. to 130° F., a penetration at 77° F. (ASTM Designation D 5–52) of 60 to 120, and a ductility at 77° F. (ASTM Designation D 113–44) of not less than 100 centimeters. The ash content shall be not more than 18 percent, as determined by ASTM Designation D 128–57.

(3) *Welding rods.*—Welding rods shall be of a type and composition approved by the contracting authority.

(d) *Payment.*—Payment for furnishing and placing metal waterstops will be made at the unit price per linear foot bid therefor in the schedule, which unit price shall include the cost of cutting, welding, placing, and coating the metal waterstops.

G–76. *Drilling Holes for Anchor Bars and Grouting Bars in Place.*—Where shown on the drawings or directed, holes shall be drilled into the rock to receive bars for anchoring concrete in.......... ---- ----- . The contracting authority will furnish *(anchor bars) and *(cement for grout). The contractor shall furnish *(anchor bars) and *(sand and water) *(all materials for grout). The dimensions of the anchor bars and the locations, diameters, and depths of the anchor-bar holes shall be as shown on the drawings or as directed. The diameter of each anchor-bar hole shall not be less than one and one-half times the diameter or greatest transverse dimension of the anchor bar specified for that hole.

Anchor bars shall be cleaned thoroughly before being placed. The holes shall be cleaned thoroughly and shall be completely and compactly filled with grout mixed in the proportions and to the consistency prescribed by the contracting authority. The anchor bars shall be forced into place before the grout takes its initial set and, where practicable, shall be vibrated or rapped until the entire surface of the embedded portions of the bars is in intimate contact with the grout. Special care shall be taken to insure against any movement of the bars which have been placed.

Measurement, for payment, of drilling holes for anchor bars and grouting bars in place will be based upon the length of holes required to be drilled beyond the surface of the excavation or rock surface. Payment for drilling holes for anchor bars and grouting bars in place will be made at the unit price per linear foot bid therefor in the schedule, which unit price shall include the cost of furnishing materials for grout, drilling the holes, and grouting the bars in place. Payment for furnishing and placing the anchor bars will be made at the unit price per pound bid therefor in the schedule. *(Payment for furnishing and handling cement will be made at the unit price per barrel bid therefor in the schedule.)

G–77. *Drainage, General.*—All drains shall be constructed at the locations shown on the drawings or as directed. Care shall be taken to avoid clogging the drains during the progress of the work, and should any drain become clogged or obstructed from any cause before final acceptance of the work, it shall be cleaned out in a manner approved by the contracting authority or replaced by and at the expense of the contractor. No pipe which has been damaged shall be used in the work if, in the opinion of the contracting authority, the pipe is unfit for use.

G–78. *Constructing Sewer-Pipe Drains With Open Joints.*—(a) *General.*—Concrete or clay sewer-pipe drains with open joints shall be placed in a bedding of pervious material which is described specifically under subsections (b) and (c) below. All sewer pipe, burlap, and material for bedding shall be furnished by the contractor. Burlap shall be 40 inches in width, 10 ounces per linear yard, in accordance with Federal Specification CCC–B–811.

The pipe shall be plain (nonperforated) concrete sewer pipe, type 1, nonreinforced, bell-and-spigot pattern, in accordance with Federal Specification SS–P–371a; or plain clay sewer pipe, standard

*Revise or delete as appropriate.

strength, bell-and-spigot type, in accordance with Federal Specification SS–P–361b, or perforated concrete sewer pipe, type 1, nonreinforced, bell-and-spigot pattern, manufactured in accordance with Federal Specification SS–P–371a, except that the pipe shall be perforated in accordance with the provisions for perforating clay pipe as set forth in ASTM Designation C 211–50; or perforated clay sewer pipe, standard strength, bell-and-spigot type, in accordance with ASTM Designation C 211–50; all as hereinafter specified. Cement used in concrete sewer pipe shall conform to cement required for structures as provided in section G–46 (Cement). Strainers and stoppers at top of drains shall be standard concrete or clay to fit pipe used.

(b) *Embankment Toe Drains.*—Plain sewer-pipe drains ______ and ______ inches in diameter shall be constructed with open joints under the downstream toe of the dam embankment and in extensions beyond the toe of the dam embankment and elsewhere, if required, as shown on the drawings or as directed. The drains shall be constructed in trenches to the prescribed lines, grades, and dimensions.

The bedding material adjacent to the pipe shall consist of natural gravel or crushed rock, or a mixture of natural gravel and crushed rock, clean and well graded from 3/16 inch to 1½ inches in size, as approved by the contracting authority, and may contain up to 10 percent of total of particles smaller than 3/16-inch size. The bedding material *(may, shall) be obtained from approved sources of coarse aggregate for concrete, *(or may be produced by screening the desired sizes from selected material from required excavations for ____________________ or from borrow pits in borrow area ______). The sand bedding to be placed in the bottom of the trench shall meet the requirements of sand for concrete. The remainder of the bedding shall consist of uncompacted pervious material. The pipe shall be laid carefully on the bedding with the bell end upgrade with joint openings not less than one-fourth inch nor more than three-eighths inch and with spigot end concentrically in the bell. *(Perforated pipe shall be laid with perforations in lower half of pipe.) The pipe shall then be covered with a minimum of ______ inches of bedding material as shown on the drawings. The bedding shall be carefully placed and tamped about the pipe so as not to disturb the pipe and to hold it securely in position while the material overlying the bedding is being placed.

Measurement, for payment, of furnishing sewer pipe and constructing embankment toe drains with open joints will be made along the centerlines of the pipe, from end to end of the pipe in place, and no allowance will be made for lap at joints or intersections. Payment for furnishing sewer pipe and constructing embankment toe drains with open joints will be made at the unit prices per linear foot bid therefor in the schedule, which unit prices shall include the cost of furnishing materials for and preparing and placing the bedding about the pipe. Measurement, for payment, of excavation for trenches for embankment toe drains will be made only to the neat lines shown on the drawings or directed, and payment for excavation of the trenches will be made at the unit price per cubic yard bid in the schedule for excavation for dam embankment foundation.

(c) *Drains for Spillway and Outlet Works.*—Perforated sewer-pipe drains ______, ______, and ______ inches in diameter shall be constructed behind the walls and under the floor of the outlet works stilling basin, and drains ______, ______, ______, and ______ inches in diameter shall be constructed behind the walls and under the floors of the spillway chute and stilling basin, as shown on the drawings or as directed.

The bedding material shall consist of natural gravel or crushed rock, or a mixture of natural gravel and crushed rock, clean and well graded from 3/16 inch to 1½ inches in size, as approved by the contracting authority, and *(may, shall) be obtained from approved sources of coarse aggregate for concrete, *(or may be produced by screening the desired sizes from selected material from required excavations for ____________________ or from borrow pits in borrow area ______). The bedding shall be placed under the pipe to minimum depths as shown on the drawings. The pipe shall be laid carefully on the *(bedding), *(lean concrete pad). The bell end of the pipe shall be laid upgrade with joint openings not less than one-fourth inch nor more than three-eighths inch and spigot end concentrically in the bell. *(The pipe shall be placed immediately after placement of the lean concrete in the pad and worked into the fresh lean

*Revise or delete as appropriate.

concrete to insure continuous support under the total length of the pipe.) Bedding material shall then be placed to provide a minimum thickness of ______ inches over the top and at the sides of the pipe. The bedding shall be carefully placed and tamped about the pipe so as not to disturb the pipe after being laid and to hold it securely in position while the overlying material is being placed. Where concrete is to be placed over or against the bedding of the drains, the bedding shall be covered with a layer of heavy burlap before the concrete is placed.

Measurement, for payment, of furnishing perforated sewer pipe and constructing drains with open joints will be made along the centerlines of the pipe, from end to end of the pipe in place and no allowance will be made for lap at joints. Payment for furnishing perforated sewer pipe and constructing drains with open joints will be made at the unit prices per linear foot bid therefor in the schedule, which unit prices shall include the cost of furnishing and placing the burlap covering for the drains and the cost of furnishing materials for and preparing and placing the bedding about the pipe. The unit prices bid for furnishing perforated sewer pipe and constructing drains with open joints shall also include the cost of furnishing materials for and constructing lean concrete pads, except that payment for the cement will be made at the unit price per barrel bid in the schedule for furnishing and handling cement. Measurement, for payment, of excavation for the pipe drains, including excavation for bedding material and lean concrete pads, will be made only to the neat lines as shown on the drawings or directed, and payment for such excavation will be made at the unit price per cubic yard bid in the schedule for the excavation for the structure for which the pipe drains are constructed.

G–79. *Laying Sewer Pipe With Calked Joints.*—Concrete or clay sewer pipe for drains ______, ______, ______, and ______ inches in diameter shall be laid with joints calked with oakum in the locations shown on the drawings, or as directed. The pipe shall be clay sewer pipe, standard strength, bell-and-spigot type, in accordance with Federal Specification SS–P–361a; or concrete sewer pipe, type I, nonreinforced bell-and-spigot pattern, in accordance with Federal Specification SS–P–371.

The pipe shall be laid carefully to the prescribed lines and grades. The ends of the pipe shall fit closely, the spigot ends shall be placed concentrically in the bells, and the joints shall be packed with oakum and thoroughly calked with suitable calking tools.

Pipe to be embedded in concrete shall be placed accurately in position, calked and held securely in place during the placing and hardening of the surrounding concrete.

The cost of furnishing all materials and embedding sewer pipe with calked joints in concrete shall be included in the unit price per cubic yard bid in the schedule for the concrete in which the sewer pipe is embedded.

Pipe not to be embedded in concrete shall be laid, sufficient suitable material shall be tamped under and about the pipe to hold it firmly in place, and after the joints are calked, additional suitable material shall be placed and thoroughly tamped about the pipe, care being taken not to disturb the pipe.

Measurement, for payment, of laying sewer pipe with calked joints will be made from end to end of the pipe in place, and no allowance will be made for lap at joints. Payment for furnishing and laying sewer pipe with calked joints will be made at the unit prices per linear foot bid therefor in the schedule, which unit prices shall include the cost of preparing an even bedding for the pipe, furnishing and tamping suitable material under and about the pipe, furnishing oakum and calking the joints, and backfilling trenches with selected material. Measurement, for payment, of excavation for trenches for pipe drains will be made only to the neat lines as shown on the drawings or directed, and payment for excavation of the trenches will be made at the unit price per cubic yard bid in the schedule for excavation for the structures for which the pipe drains are constructed.

G–80. *Drilling Drainage Holes.*—Drainage holes shall be drilled through the concrete lining of the *(spillway) and *(outlet works) tunnel(s) through the steel-liner plates or steel lagging, if encountered, and into the surrounding rock formation as shown on the drawings or as directed. The contractor will be permitted to furnish and install pipe inserts to form the holes in the concrete lining. The drainage holes shall be drilled at the angles of inclination shown on the drawings or

*Revise or delete as appropriate

as directed. All drainage holes shall be drilled to a minimum depth of ______ feet but not deeper than ______ feet as directed, from the inside of the concrete lining with approved core drills, and the holes shall be not less than 1½ inches in diameter. Drainage holes shall not be drilled until all adjacent grout holes within a minimum distance of 150 feet have been drilled and grouted.

If, after a given area has been grouted and drilled for drainage, it is found desirable to drill and grout additional grout holes, the contractor, after completion of grouting, shall open previously drilled drain holes to their original diameter and depth to provide satisfactory drainage. Opening of previously drilled drain holes by redrilling, if required, will be ordered in writing by the contracting authority, and payment therefor will be made at one-half the unit price per linear foot bid in the schedule for drilling drainage holes.

Measurement, for payment, of drilling drainage holes will be made of the depth of holes from the inside face of the concrete lining, including portions of holes formed by pipe inserts, if used. Payment for drilling drainage holes will be made at the unit price per linear foot bid therefor in the schedule, which unit price shall include the entire cost of furnishing all materials and labor to complete the work as described in this section.

G–81. ***Cast-Iron Pipe Drains.***—A ____-inch cast-iron soil pipe drain shall be installed under and through the spillway floor to drain the spillway ______ upstream from spillway station ______ as shown on the drawings. The cast-iron soil pipe, fittings, and joint material shall be furnished and installed by the contractor as shown on the drawings or as directed by the contracting authority. The spigot ends of cast-iron soil pipe and fittings shall be placed concentrically in the bells, and all joints shall be packed with tarred jute or similar material thoroughly calked with suitable calking tools so as to leave 2 inches in the bell for lead. Joints shall be poured full of molten lead in one operation. The lead shall be retained in the joints by suitable joint runners, and after the lead has cooled sufficiently, it shall be calked tightly. Pipe to be embedded in concrete shall be held firmly in position while the concrete is being placed. Materials shall conform to the following specifications:

(1) Cast-iron soil pipe and fittings, bell-and-spigot pattern, class B, Federal Specification WW–P–401.

(2) Jute calking type II (tarred), Federal Specification HH–P–117.

(3) Calking lead, type I, Federal Specification QQ–L–156.

Measurement, for payment, for furnishing and installing ____-inch-diameter cast-iron soil pipe spillway drain will be made from end to end of the pipe in place, and no allowance will be made for lap at joints. Payment for furnishing and installing ____-inch-diameter cast-iron soil pipe spillway drain will be made at the unit price per linear foot bid therefor in the schedule.

G–82. ***Dry-Rock Paving for Open Drains.***—Dry-rock paving shall be placed on a bedding of sand and gravel or crushed rock as lining for open drains as shown on the drawings. The quality of the bedding material and rock used for dry-rock paving shall be equivalent to materials for bedding for riprap and riprap. The overall thickness of the finished bedding and paving at any point shall be not less than 15 inches. The dimensions of the paving stones normal to the face of the paving, for not less than two-thirds of the surface area of the paving, shall be not less than 12 inches, and the dimensions normal to the face of the paving of any stone forming the surface of the paving shall be not less than 8 inches. The paving stones shall have an average volume of not less than one-sixth cubic foot, and not more than 25 percent of the pieces shall be less than one-ninth cubic foot in volume. Stones of the same dimensions normal to the face of the paving shall be distributed uniformly throughout the paving. Rock materials may be selected from materials for bedding for riprap and riprap furnished by the contractor. The stones shall have roughly squared and reasonably flat upper faces; the stones shall be hand-placed, with close joints, to the established lines and grades; and the spaces between the stones shall be filled with spalls and gravel or crushed rock.

Measurement, for payment, of bedding and dry-rock paving will be made on the basis of the 15-inch thickness and on the basis of the area of finished surface of rock paving in place to the lines shown on the drawings or established by the contracting authority. Payment for dry-rock paving for open drains will be made at the unit

price per square yard bid therefor in the schedule, which unit price shall include the cost of procuring, handling, hauling, and placing bedding and paving materials. Payment for excavation for open drains, to the lines and grades established by the contracting authority, will be made at the applicable unit price per cubic yard bid in the schedule for excavation for the structure involved.

F. BIBLIOGRAPHY

G-83. *Bibliography.*

[1] *Concrete Manual*, sixth edition, revised 1956, U.S. Department of the Interior, Bureau of Reclamation (for sale by the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C. $2.50.)

[2] ———, ch. VI.

[3] ———, ch. VII.

Index

U.S. GOVERNMENT PRINTING OFFICE 1960 O—484566